HEAD OF HOLYHEAD BREAKWATER.

THE RUDIMENTS OF

CIVIL ENGINEERING

By HENRY LAW, M.Inst. C.E.

INCLUDING

A TREATISE ON HYDRAULIC ENGINEERING

By GEORGE R. BURNELL, M.Inst. C.E.

SIXTH EDITION, REVISED, WITH LARGE ADDITIONS ON

RECENT PRACTICE IN CIVIL ENGINEERING

Daniel
By D. KINNEAR CLARK, M.Inst. C.E.
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Capio Lumen

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PREFACE.

The works of Mr. Henry Law and Mr. George R. Burnell on Civil Engineering, and specially on Hydraulic Engineering, first published about thirty years ago, have enjoyed an enduring reputation. But many changes, and many advances, have been made in the interval, if not so much in the principles, certainly in the practical development, of Works of Engineering; and the publishers have been induced to submit to me those treatises for revision, with instructions, at the same time, to expand and to supplement them with material embracing the most modern practice of engineers.

The range of Civil Engineering practice is indicated by the Synopsis of the Science of Civil Engineering, contained in the opening chapter of Mr. Law's Work; and, though Civil Engineers are supposed to be required to know everything comprised in that exhaustive analysis, he wisely circumscribed his work by confining his attention, for the most part, to what are specifically known as Works of Construction. He introduced, it is true, an exceptionally detailed account of the locomotive engine. Mr. Burnell, again, elaborated the scientific elements of hydraulic engineering in Chapters on Water and Air, at rest and in motion,

and on applied Chemistry. Fitting and appropriate as such discursive matter was, no doubt, to the objects of the Rudimentary Treatise of the period, I have thought it advisable, in the preparation of the present edition, to eliminate them and some other chapters of digressive matter—as well as to omit for the most part such portions of the text as had been rendered obsolete by the advanced practice of construction, or by the effacement of the subjects themselves.

Taking advantage of the very considerable excisions recommended by the lapse of time and the change of conditions, I have supplemented the original text by types of modern practice, selected for the sake of the instruction to be derived from the study of such works, as well as for the purpose of exhibiting the best practice of more recent times. I need but refer to the entirely new Chapters on the following subjects, with others:—On the Methods of Forming Foundations, on Pavements, on Railways, Tramways, Bridges, Tunnels, Sea Defences, Embankments, Breakwaters, Piers, Quay-Walls and Dock-Walls, Docks, Waterworks, Drainage of Towns—as indications of the extensive revision which the combined works of Mr. Law and Mr. Burnell have undergone, and of the entirely new matter which has been introduced. These subjects strikingly illustrate the definition of the profession of the Civil Engineer, written many years ago by Mr. Tredgold, and adopted by the Institution of Civil Engineers in their charter—"the art of directing the great sources of power in nature for the use and convenience of man."

And here it is my duty to acknowledge the aid which I have derived from the Minutes of Proceedings of the Institution of Civil Engineers, in the

preparation of many portions of the new matter; and also the courtesy of the Council, through their able and highly-appreciated Secretary, Mr. James Forrest, in supplying copies and duplicates of many of the papers and illustrations in the Minutes, for the purpose of this work.

The new matter which I have contributed, amounting to upwards of two hundred and ninety pages, is interspersed throughout the original text, and is distinguished by bracket enclosures.

D. K. CLARK.

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THE RUDIMENTS

OF

CIVIL ENGINEERING.

INTRODUCTION.

ELEMENTARY PRINCIPLES AND CONSTRUCTION.

CHAPTER I.

THE BUSINESS OF THE CIVIL ENGINEER.

THE office of the Civil Engineer consists in the designing, arrangement, and construction of all works, structures, or machines which require the immediate superintendence of a person acquainted with the principles and practice of construction.

Civil Engineering is one of those branches of knowledge which properly take their places both amongst the *sciences* and the *arts*; for a *science* consists of a collection of general principles or truths relating to any particular subject, while an *art* is the application of those principles to practice, for the purpose of effecting some particular object. The *Science* of Civil Engineering, then, informs us of the general principles of mechanics and construction, and teaches us in what way to ascertain the strains to which every part of a structure will be exposed, and of the dimensions and proportions which should be given to each, in order that they may be

able to sustain such strains without injury. And the *Art* of Civil Engineering consists in the application of these principles to the actual construction of various works, and their judicious use and modification to meet the several contingencies which arise in practice.

The duty of the Civil Engineer, embracing, as it does, almost every kind of construction, requires a very extensive and general acquaintance with most other sciences, in order to qualify him for successfully accomplishing the various works upon which he may be engaged, and of overcoming those difficulties which frequently start up unexpectedly in the progress of a work, and, but for the knowledge, talent, and perseverance of the Engineer, threaten the ultimate success of his endeavours. It is only necessary to take a glance at the list of works upon the construction of which the Engineer is engaged—Railways, Roads, Canals, Rivers, Harbours, Docks, Breakwaters, Bridges, Tunnels, and many others—to obtain at once an idea of the extent of the subjects which his knowledge ought to comprise; and further, of the immense importance of his professional labours to his fellow men.

The following classified arrangement of the several branches of Civil Engineering, with their subdivisions, will not only serve to show the extent of this science, but will guide the Engineering student in pursuing a systematic scheme in the attainment of his professional knowledge, the importance of which, both in facilitating its acquisition and in impressing it upon the memory, are too well known and too generally admitted to require any enforcement.

SYNOPSIS OF THE SCIENCE OF CIVIL ENGINEERING.

I.—MENSURATION.

1. Surveying :—(1.) Description of instruments, and their use and adjustment.—(2.) Surveying in general.—(3.) Trigonometrical surveying.—(4.) Hydrographical surveying.—(5.) Mining surveying.
2. Levelling :—(1.) Levelling instruments, their use and adjustment.—(2.) Practice of levelling.—(3.) Measuring heights with the barometer.
3. Drawing and Plotting :—(1.) Instruments for drawing and plotting, their use.—(2.) Plotting surveys, and making plans.—(3.) Plotting levels, and making sections.—(4.) Preparing Parliamentary plans and sections.—(5.) Preparing working and contract plans and sections.—(6.) Preparing detail drawings of works (bridges, &c.)—(7.) Making mechanical drawings.—(8.) Principles of projection, perspective, and shadows.
4. Estimating :—(1.) Taking out quantities from drawings.—(2.) Measuring quantities from the works themselves.—(3.) Measuring artificers' work.—(4.) Calculating, measuring, and valuing earthwork.—(5.) Estimating value or cost of works.
5. Setting out Works :—(1.) Centre lines and side widths of railways, roads, canals, &c.—(2.) Setting out bridges, viaducts, walls, &c.—(3.) Setting out tunnels and driftways.

II.—GENERAL CONSTRUCTION.

1. Statics :—(1.) Composition and resolution of pressures.—(2.) Moments of pressures.—(3.) Parallel pressures, and the centre of gravity.
2. Stability of Structures :—(1.) General conditions of stability.—(2.) Stability of polygonal framings.—(3.) Equilibrium of arches. (4.) Stability of abutments and piers.—(5.) Stability of retaining walls.—(6.) Equilibrium of suspension bridges.
3. Strength of Materials :—(1.) To resist a tensile and crushing strain.—(2.) Elasticity and elongation of bodies subject to a tensile or crushing strain.—(3.) When subjected to a transverse strain.—(4.) Elasticity and deflection of bodies subjected to a transverse strain.—(5.) To resist torsion.

4. Materials employed in Construction:—(1.) Metals.—(2.) Timber.—(3.) Natural stones.—(4.) Artificial stones, including bricks, concrete, and the various cements used in masonry.—(5.) Materials for earthwork, such as embankments, puddled banks, dams, &c.—(6.) Materials for roads and pavements.—(7.) Materials for covering roofs.
5. Different kinds of Construction: —(1.) Brickwork, —(2.) Masonry.—(3.) Forming foundations.—(4.) Carpentry.
6. Auxiliars employed in Construction:—(1.) Scaffolding, fixed and travelling.—(2.) Centerings.—(3.) Cofferdams.

III.—MECHANICS, OR CONSTRUCTION OF MACHINERY.

1. Dynamics:—(1.) Vis viva, momentum, and work.—(2.) Motion, uniform, accelerated, or retarded; gravitation.—(3.) Collision and impact of moving bodies.—(4.) Motion down inclined planes, and curves.—(5.) Motion about fixed centres; centres of percussion, oscillation, and gyration.
2. Moving Forces:—(1.) Water as a mechanical agent.—(2.) Air as a mechanical agent.—(3.) Animal strength as a mechanical agent. (4.) Heat as a mechanical agent; the steam engine.
3. Resistances to Motion:—(1.) Friction.—(2.) Resistance of the medium through which the body moves.
4. Theory of Machines:—(1.) Elements of machinery.—(2.) Teeth of wheels, racks, and pinions.—(3.) Transmission of work by machinery.—(4.) Determining the modulus of a machine in motion.—(5.) Mechanical expedients for transmitting or changing motion.—(6.) Proportioning the strength and dimensions of machinery.
5. Machines employed in Engineering:—(1.) Machines employed for transporting and raising materials, such as crabs, cranes, dredging machines, &c.—(2.) Machines employed in actual construction; such as pile-driving engines, excavating machines, pumps, diving-bells, pug and cement mills, &c.—(3.) Machines for working upon materials; such as lathes, boring, planing, mortising, riveting, and screw-cutting machines, saws, &c.—(4.) Implements and tools for excavating, boring, working in wood, metals, stones, &c.

IV.—SPECIAL CONSTRUCTION.

1. Common Roads:—(1.) Principles which should control the selection of route.—(2.) Laying out roads, and arrangement of gra-

dients.—(3.) Construction of roads.—(4.) Draining roads.—(5.) Repair of roads.—(6.) Protecting their surface by different kinds of pavement.

2. RAILWAYS :—(1.) Principles which should determine the route, and the general arrangement of the curves and gradients.—(2.) Different systems of haulage, the locomotive, the atmospheric, and the rope.—(3.) Of the general construction of the railway.—(4.) Of the permanent way, different forms of rails, switches, &c.—(5.) Of draining the line, and maintaining the slopes and permanent way.—(6.) Arrangement of termini and stations.—(7.) Construction of engines and carriages. (8.) System of working the line.

3. CANALS :—(1.) Principles which should determine the choice of the line of a canal.—(2.) Arrangement of levels, number of locks, and form of section.—(3.) General construction of canals.—(4.) Arrangement of locks, means of saving water, and obtaining feeders.—(5.) Methods of propulsion or towing, and resistance on canals.—(6.) Construction of aqueducts.—(7.) Repair and preservation of canals.

4. HARBOURS AND DOCKS :—(1.) On the construction of piers, breakwaters, and quay walls.—(2.) On the means of deepening harbours, by dredging or excavation.—(3.) Selection of site for docks, and their arrangement.—(4.) Construction and arrangement of locks; cast iron and timber gates, sluices, &c.—(5.) Construction of dock walls.

5. BRIDGES :—(1.) Selection of site, and determination of the kind of bridge.—(2.) Construction of stone and brick bridges.—(3.) Construction of iron and timber bridges.—(4.) Construction of suspension bridges.—(5.) Construction of railway viaducts.—(6.) Of forming the foundations of bridges.

6. TUNNELS :—(1.) Determination of the form and dimensions of the tunnel.—(2.) Method of excavating and securing the ground.—(3.) Sinking shafts, and driving headings or driftways.—(4.) Method of draining the tunnel.—(5.) Subaqueous tunnels.

V.—HYDRAULIC ENGINEERING.

1. HYDRAULICS :—(1.) The science of hydrostatics.—(2.) Hydrodynamics.—(3.) Pneumatics.

2. DRAINAGE AND IRRIGATION :—(1.) Drainage of open country and agricultural districts.—(2.) Improvement of outfall, and diversion of water from other districts.—(3.) Surface-draining, catchwater drains, and under-draining.—(4.) Drainage of bogs and

marsh lands.—(5.) Of *warping* up, and reclaiming lands from the sea and rivers.—(6.) Drainage of towns.—(7.) Form, dimensions, and declivity proper for sewers.—(8.) Of the collection and disposal of the sewage.

3. Supply of Water to Towns:—(1.) Principles which should guide the choice of the means of supply.—(2.) Different sources of supply: from the watershed of the country, from springs and Artesian wells, or from large rivers.—(3.) Means of estimating the quantity required, and of ascertaining the probable supply, and the quality of the water.—(4.) Systems of supply; the *constant*, or high pressure system, and the *intermittent*.—(5.) Selection of site for reservoirs.—(6.) Construction of reservoirs.—(7.) Contrivances for raising the water to the level of the high reservoirs.—(8.) Of the means of filtering and purifying the water, and of the construction of the filter beds.—(9.) Of the motion of water in pipes, and their discharge.

4. Marine Engineering:—(1.) Action of waves and currents, their modification by the contour of the shore, and the depth of water. (2.) Their action on the shore, on beaches, on vertical, sloping, and curved walls, and generally on any obstacle.—(3.) On the *régime* of coasts, and their preservation.—(4.) Construction of sea-walls, embankments, breakwaters, piers, and other structures exposed to the action of the sea, more particularly as regards their form.—(5.) Principles which should determine the selection of the site for a harbour, and the arrangement of its form.—(6.) On the causes which produce shoals and bars.—(7.) Means of keeping harbours free from such obstructions, or of removing them where already existing.—(8.) On the improvement of harbours and sea channels.

5. Improvement of Rivers:—(1.) On the tidal wave at the mouth of rivers, and its modification in passing up the river.—(2.) Principle of the conservation of tidal force.—(3.) On the antagonist agencies of the tide and land waters in rivers; and the means of determining which of these should be assisted; of the *régime* of rivers.—(4.) On the form of the shore-line of rivers, and their improvement.—(5.) Of the junction of rivers.—(6.) On the velocity of the stream, its scouring and transporting power, compared with the nature of its bed.—(7.) Effects of projections, irregularities, and obstructions, such as dams and weirs.—(8.) Of the formation and removal of shoals; their causes; of artificial scouring and sluicing.—(9.) Of the shoals formed at the mouths of rivers, their cause and prevention.

VI.—SCIENCES COLLATERALLY CONNECTED WITH ENGINEERING.

1. Somatology, or the properties of matter.
2. Chemical Philosophy.
3. Geology and Mineralogy.
4. Natural History.
5. Physical Geography, and Hydrography.
6. Mathematics.
7. Acoustics.

Of all these Sciences a certain amount of knowledge is required by the Civil Engineer; but of some more than of others, depending, in a great measure, upon those particular branches of the profession to which he may more exclusively direct his attention.

The foregoing tabular view only comprises those branches which may be said to form actually a portion of the science of Civil Engineering, but is far from including every subject with which the engineer should be conversant.

The limits, and, in fact, the object, of the present work are incompatible with a strict adherence to the above classified arrangement of the subject; and it will therefore be seen that we have omitted altogether mention of some of the matters included in the foregoing table, and that in other cases we have deviated from and modified the method of treating the subject.

CHAPTER II.

MATERIALS EMPLOYED IN CONSTRUCTION.

The principal materials made use of by the civil engineer for the purpose of construction may be classified as follows :—

1. Metals.
2. Timber.
3. Natural stones.
4. Artificial stones, including bricks, and the different kinds of mortars and cements used in masonry.

Before describing the principal properties of each of the classes of materials, it will be desirable to consider briefly the subject of their strength, and to explain the circumstances which affect the same.

Strength of Materials.

The strength of materials to resist the action of any external force to which they may be exposed varies greatly with the manner in which that force is applied ; and therefore it is necessary, in considering this subject, to divide the strength of materials as follows : first, their power to resist *extension*, or the force required to *pull them asunder ;* secondly, their power to rest *compression*, or the force requisite to *crush* them ; thirdly, their *transverse* strength, or the force required to break a bar or beam resting at each end upon supports, or which is fixed at one end and loaded at the other end ; and fourthly, their *elasticity*, or the force required to bend a bar or beam.

1st.—When any homogeneous material is exposed to a tensile strain, or a strain tending to tear it asunder, if the direction of the strain passes through the centre of the piece, its strength is proportional to its sectional area. The weight in pounds required to tear asunder a bar one inch square of metal, wood, or stone, is given in the column B in the tables of their properties given below; and the tensile strength of a piece of any other dimensions may be found, by multiplying the corresponding number in the table by the area of the piece in square inches. Thus, the weight required to pull asunder a bar of cast iron 3 inches by 4 inches would be 17,920 multiplied by 12, or 215,040 lbs.; and the weight to tear asunder a piece of white marble 1 foot square would be 551 multiplied by 144, equal to 79,344 lbs., or nearly 36 tons.

2ndly.—The experiments of Professor Hodgkinson* have shown that when a substance is submitted to a compressing force, its strength will depend upon the proportion which its height bears to its other dimensions. He found that when the height of the piece was not greater than its diameter, if round, or the length of its side, if square, its strength would *increase* as its height was diminished; but that when the height was greater than those dimensions, fracture took place by the separation of a pyramid, cone, or wedge (depending on the form of the piece), the angle of whose side was always the same for the same material, and that the strength would not vary with an increase in the height until it became equal to four or five times the diameter, when the piece would begin to bend, and its strength would then *diminish* as its height was further increased; he also found that within these limits the strength was proportional to the sectional area. The weight in pounds required to crush cubes 1 inch square of different materials is con-

* "Experimental Researches on the Strength and other Properties of Cast Iron."

tained in the columns A in the tables following; for any other dimensions, the numbers in the table must be multiplied by the sectional area in square inches; thus, the weight required to crush a block of Portland stone 1 foot square would be 1491 multiplied by 144, equal to 214,704 lbs., or 96 tons.*

* The following table exhibits the formulæ which Professor Hodgkinson has deduced from his experiments on the strength of columns; in which w is the weight in tons required to crush the column, d its external diameter in inches, d_1 its internal diameter (if hollow), l its length in feet, and a, b, c, and e are constants depending on the material of the column, and the values of which, for a few materials, are given in the second table below:—

Kind of column.	With both ends *rounded*, when the height of the column is not less than 15 times its diameter.	With both ends *flat*, when the height of the column is not less than 30 times its diameter.
Solid cylindrical columns	$w = a\dfrac{d^{3.6}}{l^{1.7}}$	$w = b\dfrac{d^{3.6}}{l^{1.7}}$
Hollow cylindrical columns	$w = c\dfrac{d^{3.6}-d_1^{3.6}}{l^{1.7}}$	$w = e\dfrac{d^{3.6}-d_1^{3.6}}{l^{1.7}}$

Material.	*a.*	*b.*	*c.*	*e.*
Cast Iron . . .	44·90	44·20	13·0	44·3
Wrought Iron . .	26·00	77·00	22·7	77·2
Cast Steel . . .	37·50	110·90	32·7	111·1
Dantzic Oak . .	1·62	4·81	—	—
Red Deal . . .	1·17	3·47	—	—

When the height of the column is less than that stated in the foregoing table, it gives way partly by flexure and partly by crushing, and the column will bear a greater weight than the table would show. In this case let w be the strength calculated from the table, w_1 the strength calculated by the rule given in the text above for the crushing strength of the material, and w_2 the actual strength of the column, then—

$$w_2 = \frac{w\,w_1}{w + \frac{3}{4}w_1}$$

[Mr. F. W. Shields gives the safe load on hollow cast-iron columns of good construction, with flat ends and with base plates.

Thickness.	Load per square inch of sectional area.	
	Length 20 to 24 diameters.	25 to 30 diameters.
inches.	tons.	tons.
$\frac{3}{4}$ and upwards	2	$1\frac{3}{4}$
$\frac{5}{8}$	$1\frac{3}{4}$	$1\frac{1}{2}$
$\frac{1}{2}$	$1\frac{1}{2}$	$1\frac{1}{4}$
$\frac{3}{8}$	$1\frac{1}{4}$	1]

3rdly.—Let Fig. 1 represent a bar or beam of any material, resting at each end on two fixed supports A and B, and having a weight W suspended from the centre C. Now the amount of W, or the weight which will be required to break the beam, when applied in the manner here described, will be directly proportional to the breadth of the beam multiplied by the square of its depth CD; or, what is the same, to its sectional area at C, multiplied by its depth, and inversely proportional to the distance A B between the supports. The numbers in the column C in the tables following show the weights in pounds required to fracture bars of the several materials 1 foot long and 1 inch both in breadth and in depth, the weight being applied in the centre of the bar. To find the weight in pounds required to fracture a piece of any other dimension, we must multiply the number in the table by the square of the depth in inches, and by the breadth in inches, and we must divide the product by the distance between the supports in feet; thus, suppose the distance A B is 10 feet, the depth of the beam 6 inches, and its breadth 4 inches, the material being cast iron, then 2045 multiplied by 36 and by 4 and divided by 10 will give 29,448 lbs.

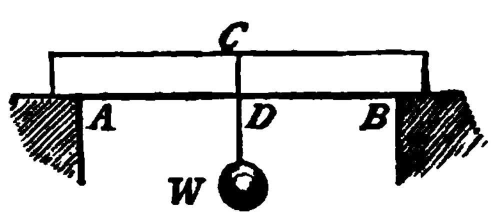


Fig. 1.

for the weight which being applied in the centre of the beam would break it.

When the weight, instead of being suspended from the centre of the beam, is distributed or spread equally over it from A to B, it will support just double the load; that is, twice the weight will be required to break it when thus distributed which would be required if suspended from the centre.

If the beam, instead of being supported at each end, is only fixed at one end A, Fig. 2, and has the weight suspended from the other end B, it will only bear one-fourth of the weight which it would do if supported at each end and loaded in the middle. In this case, also, if the weight be distributed equally, the beam will support twice as much as if it were suspended from the end.

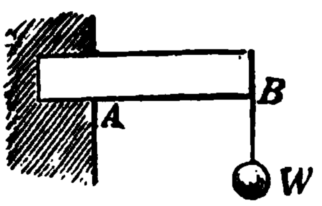


Fig 2.

In all the cases which we have considered above, the form of the cross section of the beam has been supposed to be rectangular, as in G, Fig. 3. This form of section, however, is not the strongest which could be chosen; for, by altering it, the same quantity of material may be made to sustain nearly three times the weight.

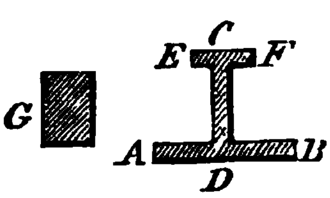


Fig. 3.

A form recommended by Professor Hodgkinson, and which has been very generally adopted in practice, is shown in Fig. 3. The weight in pounds which would be required to be applied in the centre to break a beam of this form, supported at each end, will be found by multiplying 4852 by the area in square inches of the lower flange or web A B, and by the depth C D in inches, and dividing the product by the distance between the supports in feet.

[Since Mr. Hodgkinson's rule was promulgated, it has been perceived that though the experimental basis of the deductions

arrived at by Mr. Hodgkinson was sound so far as it went, yet that his proportions did not sufficiently provide for practical necessities. It is rarely now that cast-iron beams are made of the extreme proportions of section advocated by Mr. Hodgkinson. They are, on the contrary, made with the upper and lower flanges more nearly equalised in area, and the flanges and the web more nearly equalised in thickness, in order that a sound casting may be made, and a beam of a more convenient section may be produced. A cast-iron beam should have the sectional area of the upper flange not less than one-half of the lower flange, and the thickness of the web should be about two-thirds of that of the lower flange.

Approximate rule for the transverse strength of a cast-iron beam.—Let the ultimate tensile strength of the metal be 7 tons per square inch. To the sectional area of the lower flange add a fourth of the sectional area of the web, calculated on the total depth, both in inches, multiply the sum by the total depth in inches, and by $2\frac{1}{3}$, and divide the product by the span in feet. The quotient is the breaking weight at the middle, in tons.

For any other tensile strength, use it as the multiplier instead of $2\frac{1}{3}$, and divide the product by 3 and by the span. The quotient is the breaking weight.

Rolled wrought-iron flanged beams or joists, having equal flanges, are much employed in buildings.

Approximate rule for the transverse strength of solid wrought joists of ordinary proportions.—To the sectional area of one flange add one-fourth of the sectional area of the web, calculated on the total depth, both in inches; multiply the sum by the depth in inches and by 133, and divide by the span in feet. The quotient is the breaking weight in hundredweights applied at the middle.]

4thly.—When a beam, supported as in in Fig. 1, has a weight suspended from its centre, it is bent into a curved

form, and the distance that the middle point C of the beam is drawn down below its former position, is called the *deflection* of the beam. The amount of the deflection is directly proportional to the weight applied, multiplied by the cube of the length A B, and is inversely proportional to the breadth of the beam multiplied by the cube of its depth; it may be determined, for any particular case, by multiplying the cube of the length in feet by the weight in pounds applied in the centre, and dividing the product found against the material of the beam in column D of the following tables; multiplied by the breadth and the cube of the depth, both in inches, the quotient will be the deflection, also in inches. Thus, suppose a beam of English oak 10 feet in length, 9 inches in depth, 6 inches in breadth, and loaded with 5,000 pounds in the centre, what will be the deflection? In this case, 1,000 multiplied by 5,000 equals 5,000,000, and 3,369 multiplied by 6 and 729 equals 14,736,006; then 5,000,000 divided by 14,736,006 equals ·34 inch, the deflection required.

General Properties of Metal.

Name of Metal.	Weight in lbs.				Strength.				
	Of a cubic foot.	Of a plate 1 ft. square and 1 in. thick.	Of a bar 1 in. square ... ng.	... n. in diam. and 1 ft. long	Weight in lbs. required to crush 1 square inch. (A)	Weight in lbs. required to tear asunder 1 square inch. (B)	Weight in lbs. required to break a bar transversely 1 in. sq. and 1 ft. long (C)	Multiplier for elasticity. (D)	Expansion in length for 1 degree of heat.
Cast iron	450	38·4	3·	51	107,750	17,920	2,045	42,593	·00000617
Wrought iron	475	40·0	3	31	—	58,952	2,290	57,885	·00000698
Steel	490	40·8	3·	37	—	80,000	—	67,129	·00000636
Copper (cast)	549	45·7	3·	39	116,480	19,072	—	—	·00001430
Gun-metal	510	42·5	3·	78	—	35,840		22,854	·00001009
Brass (yellow)	528	43·6	3·	35	158,520	17,958	890	20,671	·00001044
Lead (cast)	710	59·3	4·	36	7,840	1,824	199	1,667	·00001593
Zinc (cast)	439	36·6	3·	50	—	—	746	31,667	·00001634

Cast iron may be divided into two varieties, the *white cast iron*, which has a white and radiated or crystalline appearance when broken, and is hard and brittle; and the *gray cast-iron*, which has, when fractured, a gray colour, granular texture, and metallic lustre, and is very much softer and tougher than the white. Between those two a great number of intermediate varieties exist, so that we may pass from one to the other by almost imperceptible gradations. The best practical test of the quality of cast iron is by a blow with a hammer on one of its edges, which, if the iron belongs to the first variety, will break off small particles; but if to the second, will only indent it. It is much used for columns, for which, from its great power of resisting compression, it is peculiarly adapted.

It has also been much employed for beams or girders, although wrought-iron has been substituted for it. Steel is now used to a considerable extent in the manufacture of railway rails, bridges, and other works where the qualities of lightness, strength, and toughness are required in combination.

General Properties of Timber.

Name of wood.	Weight in pounds. Of a cube foot.	Weight in pounds. Of a bar 1 inch square and 1 foot long.						
Ash	48	·33	—	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Beech	44	·30	—	11500	519	3133	27	44
Chestnut	55	·38	—	8100	—	2140	37	44
Elm	35	·24	1284	9740	338	1620	32	44
Fir, Mar Forest	44	30	—	6900	407	1840	—	—
„ New England	35	·24	—	10210	367	5072	—	—
„ Riga	47	·33	—	9500	369	2684	20	75
Larch	34	·24	4920	12240	330	2094	33	45
Mahogany, Honduras	35	·24	—	11475	—	3690	72	40
„ Spanish	53	·37	—	7560	-	2096		
Norway spar	36	·25	—	8320	491	3374	15	60
Oak, Adriatic	62	·43	—	12830	461	2256	—	—
„ Canadian	55	·38	—	10220	589	4863	34	53
„ Dantzic	47	·33	—	12720	486	2757	—	—
„ English	58	·41	3860	11880	557	3369	32	42
Pine, pitch	41	·29	—	9800	544	2837	—	—
„ red	41	·29	—	11840	447	4259	—	—
Sycamore	38	·26	—	9620	—	2400	29	32
Teak	47	·32	—	12920	821	5589	—	—

Of different kinds of timber, oak, chestnut (when exposed to a free circulation of air), cedar, larch, and mahogany (when kept dry) are amongst the most durable. Beech, alder, and elm are very durable when *constantly* immersed in water or wet ground, and are therefore well adapted for the piles, &c., for foundations. When exposed, however, to the weather, or in situations where they are alternately wet

and dry, they are soon found to decay, as are also ash and mahogany. Beech, alder, and sycamore are very liable to the attacks of worms. Oak and larch are the best woods for resisting decay when exposed to the weather; but they are both liable to split and warp in seasoning, especially oak. Mahogany warps and splits in seasoning less than any other wood. Elm and larch bear the driving of nails or bolts best, being less liable to split than any other timber.

There are two different kinds of decay to which timber is liable, namely, the *wet* and *dry rots*, both of which arise from the same origin, the fermentation and consequent putrefaction of the *albumen* or *sap*, caused in one by alternate exposure to wet and dry, and in the other by the want of a free circulation of air round the timber. Both these kinds of decay arising from the presence of the sap, it is of importance to lessen its quantity as much as possible, with which object timber should be either felled in the winter months of December, January, and February, or if in summer, in July, at which seasons the sap exists in the tree in much smaller quantities than at others. It is also desirable, after the timber has been felled, to remove whatever sap may be in it as speedily as possible, which process is termed *seasoning* the timber, and is effected either by simply exposing the tree after stripping off its bark to the air, taking care to protect it from the weather, by which the moisture and sap are gradually evaporated; or by a process termed *water seasoning*, which consists in immersing the timber for about a fortnight in a stream of pure running water, by which the sap is extracted and dissolved, and afterwards gradually drying the timber.

Various processes have been patented for preserving timber both from rot and from the attack of worms. Of these, Kyan's consisted in immersing the timber for a period varying from seven to fourteen days (according to the size of the timber) in a solution of corrosive sublimate. By Payne's

process the timber is enclosed in a close iron vessel, in which a vacuum is formed by the condensation of steam, assisted by air pumps; a solution of sulphate of iron is then admitted into the vessel, which instantly insinuates itself into all the pores of the wood, previously freed from air by the vacuum, and, after about a minute's exposure, impregnates its entire substance; the sulphate of iron is then withdrawn, and another solution, of muriate of lime, thrown in, which enters the substance of the wood in the same manner as the former solution, and the two salts react upon each other, and form two new combinations within the substance of the wood—muriate of iron and sulphate of lime. One of the most valuable properties of timber thus prepared is its perfect incombustibility: when exposed to the action of flame or strong heat, it simply smoulders and emits no flame.

[Generally speaking, if the solutions of mineral salts be used of sufficient strength, and the process be continued long enough to coagulate all the albumen, decay will be retarded for a very long period, but still the fibre of the timber is left unprotected. Now, unless the fibre is also acted upon, the process of preparation must be incomplete. Impressed by such considerations Mr. Bethell sought for some antiseptic, which, being injected into the pores or capillary tubes of the timber, should bring it into a condition similar to that of the mummies and mummy cases in Egypt, which were prepared by saturating them with the petroleum or mineral pitch found floating on the Dead Sea. It has been proved by experiment that oil of tar, or creosote, is the most powerful coagulator of the albumen, whilst it, at the same time, furnishes a waterproof covering for the fibre, and by its antiseptic properties prevents putrefaction. Mr. Bethell found that by forcing at least 7 lbs. of creosote oil into each cubic foot of timber the production was perfect.

It is usual to specify for the injection of 8 lbs. of creosote

oil per cubic foot of timber for railway sleepers and other purposes. The creosoting process is that which is generally followed for the purpose of preserving timber.]

General Properties of Natural Stones.

In the above table, the values are the averages of observations made, in the case of the sandstones, upon stone from the quarries of Craigleith, Darley Dale, Hedden, and Kenton; in the case of the oolites, from the quarries of Ancaster, Bath Box, Portland, and Kelton; in the case of the limestones, from the quarries of Barnack, Chilmark, and Ham Hill; and in the case of the magnesian limestones, from the quarries of Bolsover, Huddlestone, Roach Abbey, and Park Nook. These observations were made by the Commission appointed to examine and report upon the best stone to be employed in the new Houses of Parliament, and on their recommendation the magnesian limestone was selected for that purpose.

The values in column A in the above table are those under which the stone first begins to crack; the next column contains the weight required absolutely to *crush* the stone: the

first is that which ought to be considered, practically, as the crushing weight. The seventh column gives the weight of the stone detached by Brard's process, and may be looked upon as expressing the relative power of the weather and atmosphere upon the stone.

General Properties of Artificial Stones and Cements.

Bricks may be regarded as artificial stones, formed by moulding prepared clay into the required form and then burning the same in a kiln. The quality of bricks varies greatly according to the nature of the earth from which they are made, the care taken in their manufacture, and being more or less perfectly burnt. The weight required to *crush* a square inch of brick varies from 1,200 lbs. to 4,500 lbs., but about half the crushing weight will produce fracture in the brick. The weight of a cubic foot of brickwork, set in mortar, is about 117 lbs., and in cement about 125 lbs. The tensile strength of bricks is somewhere about 275 lbs. for every square inch, but in construction they are seldom, if ever, exposed to a tensile strain. Great care should be taken in the choice and selection of bricks for structures exposed to the weather or to the action of water; in such situations, only the hardest-burnt and best-made bricks should be employed.

All kinds of mortars and cements consist of lime (a metallic oxide) combined with other substances, such as sand, gravel, clay, &c., the quality of the mortar depending upon the proportions of these ingredients, as also upon the skill with which it has been prepared. Lime is obtained by submitting limestone, which is a carbonate of lime, to a high temperature, by which the carbonic acid is driven off, and the lime left in a pure state, or only united with certain proportions of other earths and oxides. This process is termed *calcination*, and requires to be conducted with care, to ensure the

whole of the carbonic acid being expelled without fusing or vitrifying the limestone. The lime, after being burnt, should not be exposed for any length of time to the air, from which it would re-absorb carbonic acid gas and water, and would be slowly reconverted into carbonate of lime. The next process is that of *slacking* the lime, which consists in pouring over it a certain quantity of water, with which it immediately combines, the water being rapidly absorbed, with the generation of considerable heat and large quantities of vapour, and the lime falling into an impalpable powder, which is a chemical compound of water and lime, termed *hydrate of lime*. The same care should be taken not to expose slacked or unslacked lime to the air, from which it would, in the same way, absorb carbonic acid gas,

The hydrates of lime obtained by the process above described differ greatly in their properties, according to the composition of the limestone from which they have been obtained. The pure limestones yield a lime, termed by builders *rich* or *fat*, the principal properties of which are, that it will slack with great facility, absorb a very large quantity of water, with the generation of very great heat and a considerable enlargement of bulk; and then, when kneaded into a paste and immersed in water, it will remain in its soft state for years, and in running water would be entirely dissolved. Those limestones, however, which contain a large quantity of silica and alumina* yield a lime termed *hydraulic*, from its property of hardening under water; they slake with much greater difficulty than the rich limes, require less water, occupy a longer time, and do not undergo so great an increase in bulk; but their most important quality is, that when made into a paste and im-

* Silica is an acid formed by the union of oxygen with the metal *silicon*, and is the principal ingredient in sand. Alumina is a metallic oxide, composed of oxygen and the metal aluminium, and is the base of clays.

mersed in water, they *set*, or become solid, in a few days, and, at the end of a year or less, attain such a degree of hardness as to splinter under a blow, and are then perfectly insoluble in water. Between these two there are a great variety of limes possessing intermediate properties. The hydraulic properties of the latter kind appear to be owing to the presence of a certain proportion of clay, and it has been found that, by mixing clay with the richer limes and burning them together, an artificial hydraulic lime or cement may be produced possessing the same properties; and some of these attempts have been attended with considerable success.

Mortar is prepared by kneading the lime into a paste with water, and adding certain quantities of sand, very fine gravel, or puzzuolana,* determined by the quality of the lime, and the purposes to which the mortar is to be applied.

Roman cement is a species of hydraulic lime, prepared by calcining stones or boulders of septeria, picked up on the sea-coast, principally in the neighbourhood of Harwich, and the Isle of Sheppy. The stones, when calcined, instead of being slacked, are ground in a mill to a very fine powder. This cement possesses the invaluable property of *setting* under water in a few minutes. It is frequently used quite pure, or without the admixture of any sand, in situations where rapid setting is a matter of importance.

Concrete is composed of lime or of cement, mixed with from four to seven or eight times its bulk of sand, gravel, broken stone, &c., the proportions depending upon the purpose for which it is used. It should be thrown from a considerable height, by which its solidity is greatly increased.

The chemical action of salt water upon materials immersed in it, and the peculiar ravages to which some of these are

* Puzzuolana is a pulverulent volcanic earth, found at Puzzuoli, near Naples, and is principally composed of silica and alumina.

exposed from members of the animal kingdom, are deserving of notice.

Some stones and mortars, not only when immersed, but also when exposed to the sea-air, may often be noticed to decompose and to become covered by an efflorescence of the carbonate of soda, resulting from the action of the hydrochloride of soda in suspension in the atmosphere, or in combination with the water, upon the carbonate of lime. The hydrochlorides of magnesia present in sea-water act in a very peculiar manner upon some stones and mortars; for when the former exist in a state of protocarbonates of lime the magnesia enters into combination with it, and as during that process a new crystalline arrangement takes place, it is frequently the case that the stone disintegrates. With the argillo-calcareous stones, however, this action does not take place, and it would thus appear that the combination of the lime with the alumina is sufficiently energetic to enable the stones in which that state prevails to resist the decomposition of the sea-water. The same remarks apply to mortars and cements; for it is found that unless the mortars made with ordinary limes are perfectly carbonised before being immersed, or unless the cements be obtained from natural argillo-calcareous rocks, or, if artificial, unless the lime and alumina have been made to combine intimately by the effects of fusion, however well they may appear to resist in the commencement, they will eventually be certain to disintegrate. At Algiers, Brest, Cherbourg, and the Ile de Rhé, some mortars were employed for the formation of large blocks of concrete, and were composed of moderately hydraulic limes mixed with artificial puzzuolanas, prepared, in accordance with Vicat's suggestion, merely by exposing clays to a low heat in such a manner as to allow free access of air to all the parts in incandescence. The concretes thus made resisted satisfactorily for some time, but at the expiration of two or three years they fell to powder; whilst in all cases where

the natural puzzuolanas have been employed they have not yielded. It appears, therefore, that there are certain changes produced in the alumina by the action of intense heat which render it more capable of combining with lime; and it is probably in this manner that we may account for the admirable results obtained by the application of the Portland cement.

Particular stones, however hard and polished they may be, and in spite of the incessant action of the waves, become rapidly and almost entirely covered with shells and seaweed in certain positions, whilst in others they are left bare. This also is true with respect to some mortars; for blocks of concrete, which have only been immersed for ten days, have been noticed to be covered with marine plants. The boring mollusca frequently attack the softer limestones, with sufficient rapidity to render it necessary to exercise great caution in the choice of the materials employed within the range of their destructive energies. Granites and silicious sandstones are free from their attacks, and some descriptions of limestones enjoy the same immunities; but the precise nature of the latter class of stones is not known with any tolerable degree of certainty. The animals exercising this action upon stones are of the tribe of the *Lithodomi*, or more frequently, in our seas, of the *Saxicava rugosa* and the *Pholas*, the latter attacking principally the chalk, or other pure and soft carbonates of lime.

Iron, whether in the water or only exposed to its vapours, corrodes with great rapidity, wrought iron decaying, as might be expected, more rapidly than the cast metal. Painting or galvanising does not appear to retard the destructive chemical action of the salt water materially, in whatever state the iron may be; but there would appear to be a specific difference in the nature of the action upon the cast from that upon the wrought iron, for it is found that the latter becomes simply oxidated to a greater or less depth,

whilst the former, after an immersion for about thirty years, becomes converted so thoroughly into a carburet of iron, closely resembling the plumbago of commerce, that it may be easily cut with a knife. De Cessart mentions that, in removing some works executed by Vauban a century previously, he found that in many instances the wrought-iron bolts were intact, whilst other bolts, inserted in precisely analagous positions at a subsequent period, had corroded within a very few years. There would, therefore, appear to be some peculiar states of the iron as employed in the arts which modify its powers of resistance to the chemical action of the salt water. The greatest practical inconveniences attached to the chemical action of the sea-water upon iron are—firstly, that its powers of resistance are diminished; and secondly, that as its bulk diminishes also, especially when oxidation takes place, the play thus superinduced upon the framing it is intended to strengthen becomes very great.

Copper and gun-metal oxidate in salt water to a very insignificant depth. They do not appear to be otherwise affected, nor do they lose their powers of resistance. If, by means of any description of paint, or of other preservatives, the oxidation of the exposed surfaces be prevented, these metals are frequently found to be covered with shells or marine plants.

The most important observation to be made with respect to the employment of metals in sea-water is, that under no circumstances should any two different kinds be employed in contact with one another. In such cases a galvanic action takes place by the intervention of the salt water, which produces very rapid and important chemical decomposition.

If wood be kept constantly under water it is found that it will last for an indefinite period, and that in the parts left alternately wet and dry a collection of marine plants and shells, especially mussels, is rapidly formed. The principal

danger to which wood is exposed in our seas is, however, that caused by the ravages of a species of worm called the *Teredo navalis*. It is said that this worm is a native of India, and that it was introduced to Holland some 200 years since, from whence it has spread through the ports of Northern Europe. As the fossil wood of the Isle of Sheppey is frequently bored by these worms, whose casts are preserved in the fossil state equally with the wood itself, it may fairly be questioned whether the above story can account for the existence of these pests. Be this as it may, it is not the less the case that the teredo bores into the heart of the wood, and destroys the strongest carpentry with frightful rapidity. Thus at Dunkirk wooden jetties are so speedily eaten away that they require renewal every twelve or fifteen years; at Havre a stockade was entirely destroyed in six months; at Lorient wood only lasts about three years in the sea-water; and at Aix the hull of a stranded vessel was found to have lost half its weight in six months from the ravages of these animals. On our own coasts the same destruction is caused by this apparently insignificant enemy; at Southampton, Ryde, Brighton, Dover, &c., the teredo has destroyed jetties with equal rapidity to that observed on the French coast, as above cited.

When the teredo enters a piece of wood it is so small as not to leave any perceptible trace of the passage by which it entered; subsequently it increases until the bore of the passage it occupies is equal in volume to the little finger. It only attacks the interior of the wood it enters, and oftentimes the latter will break off before any external indication is given of the presence of the worm. In piles or other works in the sea, the zone most affected is that immediately below the main level of the sea; occasionally the teredos extend their ravages below the line of low water of the equinoctial tides, but they rarely mount higher than the line of high tide at neaps. It is believed that they cannot

exist under mud so compact as to exclude air; and there are some local irregularities in their distribution hitherto unaccounted for; that is to say, they are often found in some parts of a roadstead or harbour, and not in others.

Engineers have endeavoured to prevent the ravages of these creatures upon jetties or fascine banks, by either covering them with nails or by sheeting them with copper, by coating them with verdigris or cement, or by impregnating the wood with some saline solution. Of these methods, that of covering the exposed surface of the wood with nails, about $\frac{1}{2}$ inch square at the head, appears to answer the best; but in spite of all the care and attention with which it may be performed, its successful results are always problematical. Mr. Hartley, of Liverpool, asserts that the green heart wood of Demerara is not subject to the attacks of the teredo, and the Sabicu wood, from the same colony, is said to possess the same property; but these are the only known exceptions to the rule. All other woods—oak, teak, fir, elm, alike—whether hard or soft, yield rapidly.

There are also other small worms, which do not attack the heart of the wood, like the teredo, until, at least, they have destroyed all the outer parts. Their ravages are, to a certain extent, combated by covering the outside of the wood by thin slabs of the same description, which are removed as soon as they are eaten, and replaced by others.

[The creosoting process already noticed has been found to afford the most efficient means of preserving timber, whether on land or in water.]

CHAPTER III.

DIFFERENT KINDS OF CONSTRUCTION.

Brickwork.

There are two different methods of building brickwork, depending upon the relative position in which the bricks are placed. When a brick is laid with its end appearing upon the face of the wall, as A, Fig. 4, it is then called a *header*, and when with its side as B, it is then called a *stretcher*. Each horizontal layer or stratum of bricks in a wall is termed a course, and it should be so built that the vertical joints between the bricks of one course are not in the same line with those of the course above or below it; thus in the figure the joint C has no joint above or below it, but solid bricks; when the bricks are so arranged, they are said to *break joint* or *bond* with each other. There are two different methods of bonding walls in very general use, namely, *old English* bond, which consists in laying a course of headers and then a course of stretchers, as in Fig. 4;

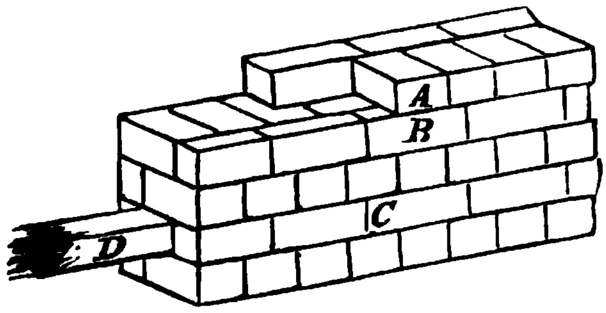


Fig. 4.—English Bond.

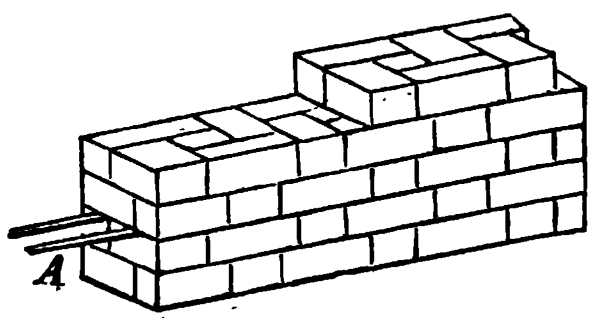


Fig. 5.—Flemish Bond.

and *Flemish* bond, which consists in laying, alternately, headers and stretchers in each course, as in Fig. 5. The

Flemish bond has the neatest appearance upon the face of the wall, but is much inferior to the old English bond in strength, and also requires much more cutting of the bricks.

Where it is requisite that the wall should be of considerable strength, the bond of the bricks only is not always sufficient; on such occasions it was customary to build a piece of timber into the wall, as shown at D, in Fig. 4, which ran through its whole length. This method, however, of bonding walls is very uncertain, because the strength of the wall depends upon the timber continuing in a sound state; and should it rot, as in such a situation it is very likely to do, we have perhaps no means of ascertaining the fact, and are only made aware of it by the failure of the wall. This method of bonding is in consequence almost entirely superseded by the *hoop-iron* bond, first introduced by Sir Isambart Brunel, and which consists in laying hoop iron flatwise between the courses, as shown at A in Fig. 5. The iron should be slightly rusted, which greatly increases its adhesion to the cement or mortar.

MASONRY.

In the construction of masonry, the same precautions are adopted as in brickwork, so to dispose the vertical joints that the wall may have a sufficient bond; and this may be easily effected, since the size of the stones is not fixed. In the neighbourhood of the quarries, where rough stone is plentiful, it is frequently employed in its rough state, without being faced or reduced to square dimensions, and is then termed *rubble* masonry. Figure 6 represents a wall built of rubble, but having the coping (A B), the plinth (C D), the quoin (B D), and the piers (A C), constructed of cut stone, which gives solidity to the wall and adds to its appearance.

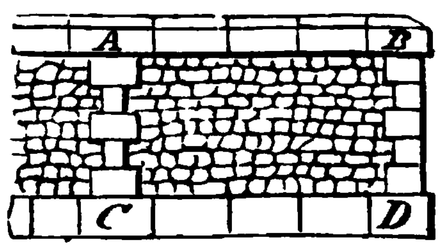


Fig. 6.—Rubble Wall.

When a wall of masonry is of any thickness, it is frequently cased with cut stone on both sides, the middle being filled in with rubble; in such cases, *heading* or bond stones, A A, Figs. 7 and 8, should be carried entirely through the thick-

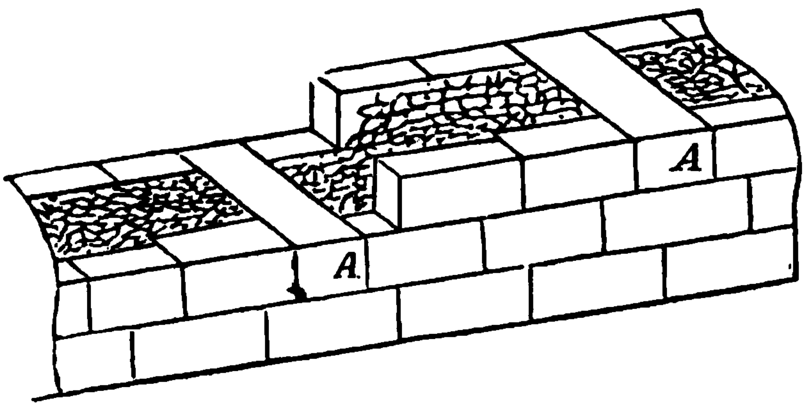


Fig. 7.—Cased Rubble Wall.

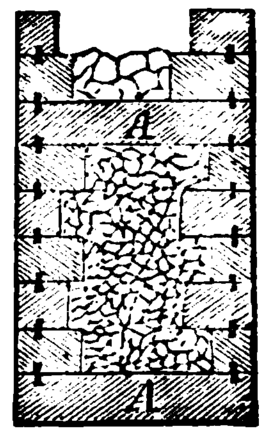


Fig. 8.

ness of the wall at certain intervals, to prevent the sides being forced apart by the settlement of the rubble between them.

In cases where it is necessary that the stones should not slip upon each other, and also to prevent the joints from separating, it is usual to insert between them pieces of iron or copper of a dovetail form, as shown at A, Fig. 9, which are termed *cramps* or *dowels*; these are inserted half in each stone; and the two having been placed, lead is run into the space round the dowel, which fixes it firmly to the stone. Dowels are sometimes made of slate or other hard stone, and are then run with cement.

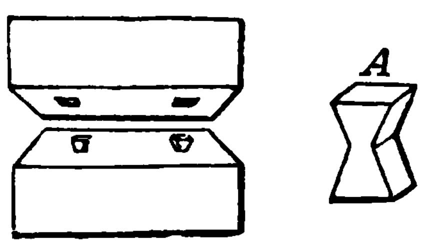


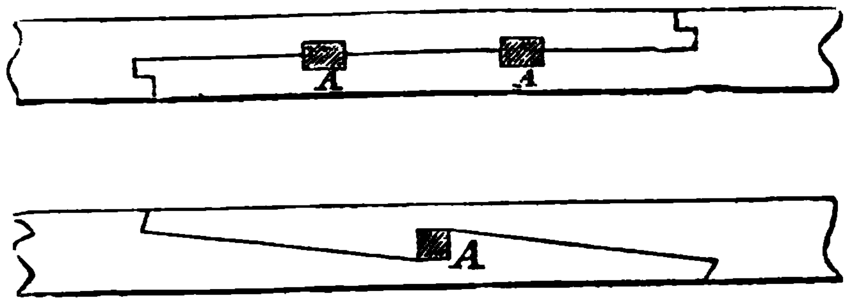
Fig. 9.

Carpentry.

The most important branch of carpentry to the civil engineer is that which relates to the methods of joining or connecting timbers together; and we shall briefly describe those most usually employed.

Figs. 10 represent two different methods of joining two pieces of timber in the direction of their length: such a

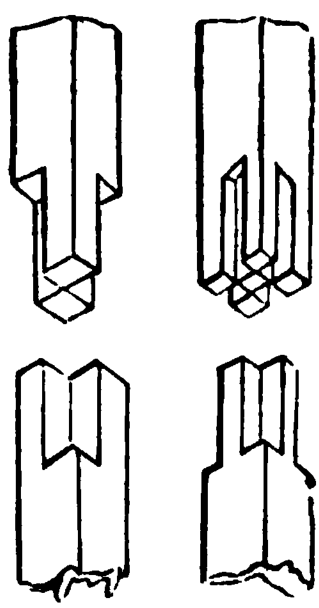
joint is termed a *scarf*. A A are wedges very slightly tapered, termed *keys*. These keys should not be driven in



Figs. 10.

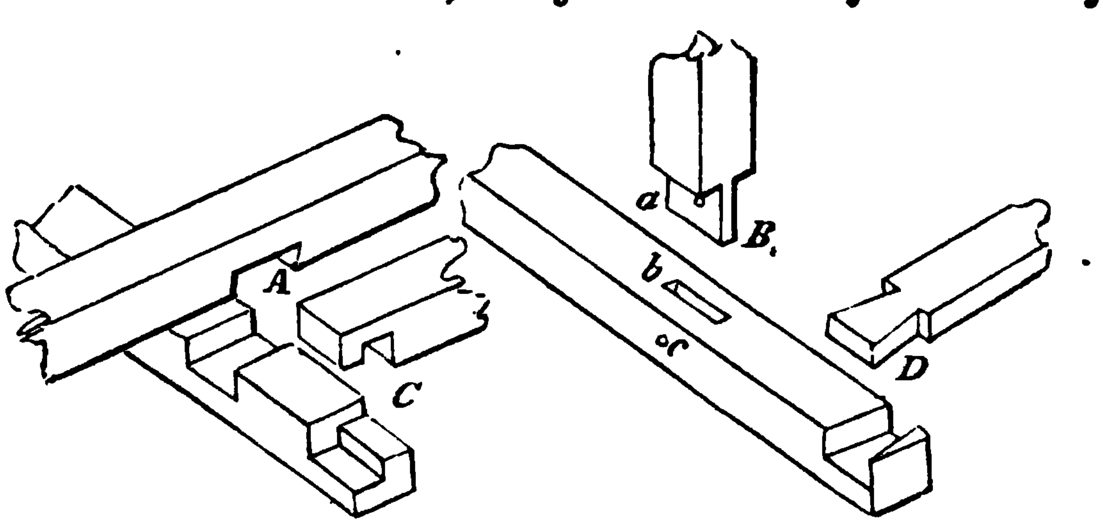
tighter than is sufficient to bring the parts together, and not so as to cause any strain on the joint. It is usual, where great strength is required, to secure the joints with plates and bolts of iron.

When it is desired to lengthen a timber placed vertically, as a post, it may be done in either of the ways shown in Figs. 11, of which the left-hand figure is the simplest.



Figs. 11.

When two timbers cross each other at right angles, it is usual to notch each of them half through as at A, Figs. 12, which is termed *halving* them. When one timber merely meets or abuts against the other, the joint is formed, as shown at B, Figs. 12, which is termed *mortising* them together; the tongue *a* is called a *tenon*, and the hole to receive it *b* the *mortise*; the joint is usually secured by an

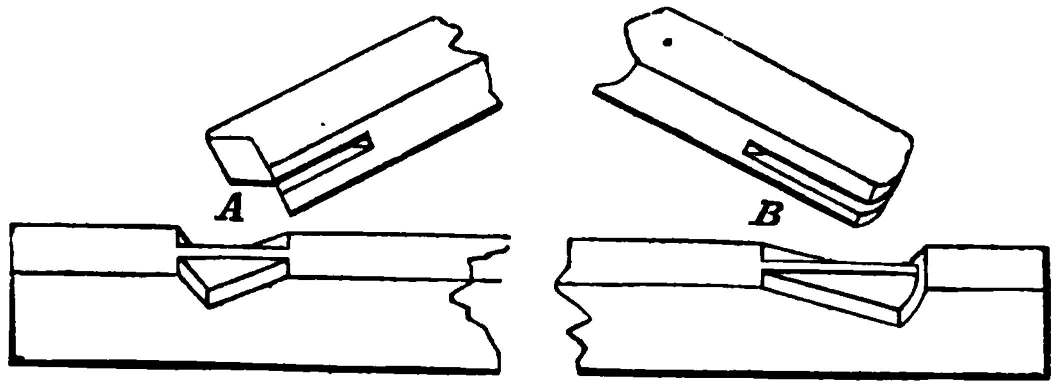


Figs. 12.

oak pin driven in at *c*. When both pieces meet, forming a

right angle or corner, they may either be *halved* together as shown at C, or *dovetailed* as at D; the former is the best, as being less affected by shrinkage of the wood.

When one timber abutting against another makes an acute angle with it, as in the case of the principals of a roof, the joint may be formed as shown at A, Figs. 13; where,



Figs. 13.

however, there is a considerable strain upon the joint, it is better to make it as shown at B, in which the bearing is more equal, and is not affected by any settlement of the framing.

[All timber may be classed under two heads, namely, hard-wood and soft-wood. These two classes of timber require very different kinds of treatment. Hard-wood, as its name implies, is hard, and it is generally brittle. A hard-wood beam may be loaded with scarcely any deflection almost to the breaking point, and it will often break when overloaded without giving any previous notice of fracture. A soft-wood beam, on the contrary, will deflect so much as to render it useless before the breaking point is reached, and it possesses more elasticity and is much lighter than a hard-wood beam.

Hence, whilst hard-wood is best suited for piles, uprights, and capsills supported at short intervals; soft-wood is better for the chords of timber bridges and for trussed beams to which a camber is given.

In hard-wood a considerable degree of dependence may be placed upon the strength of the tenons; but in soft-wood

the reverse is the case, and a different system of joints should be employed.

The use of bolts in carpentry is to hold the abutting portions of timber to their work, not to take a cross strain. In a great many cases hard-wood dowels might be substituted for iron bolts with advantage. It is true that a screw-bolt gives great facilities for drawing timbers together taut which have shrunk from their original position; but the same thing may be done by using a screw clamp and wedging up the ends of the dowels.

Straps are inferior to bolts, and they are much more expensive. Thus, in the case of a raking strut abutting on a horizontal beam, a bolt passed through both, as in Fig. 14, will hold the strut in place for a time, even if the timbers have shrunk from their bearings; whilst in the case of a strap used for the same purpose, there is nothing to prevent its slipping out of place in the event of the shrinking of the timber. See Figs. 15 and 16.

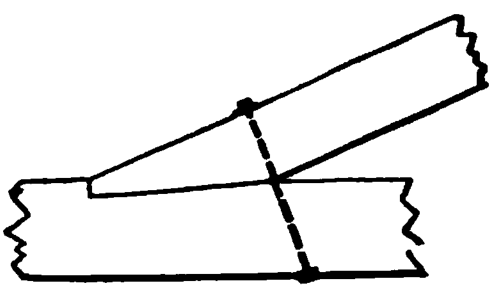


Fig. 14.

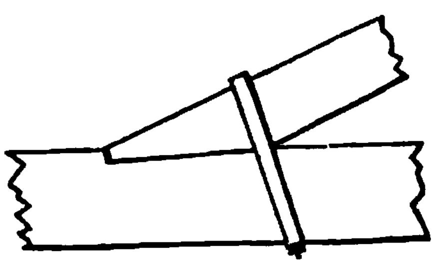


Fig. 15.

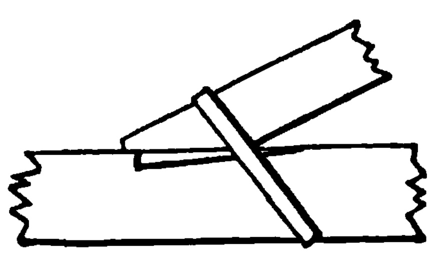


Fig. 16.

Angle-bands, Figs. 17 and 18, upon framing are always objectionable. They are wrong in principle, as they do not

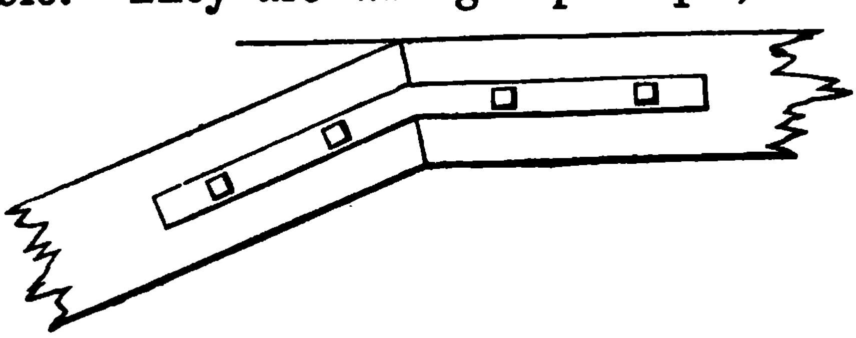


Fig. 17.

allow for the shrinking of the timber. In the case, for instance, of a strut abutting on a straining pier, secured by

an angle-band, when the timber shrinks—and it is sure to do so in time—the pressure is transferred from the ends of the timber to the bolts which fasten the bands. The

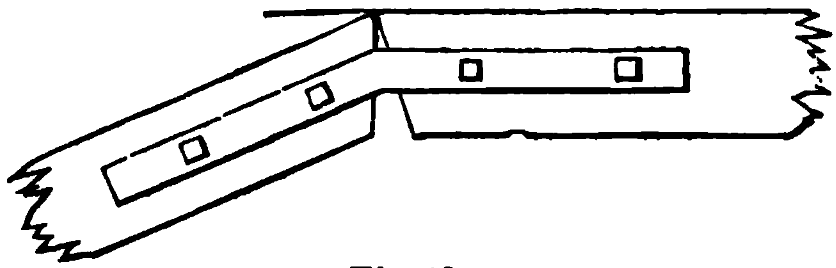
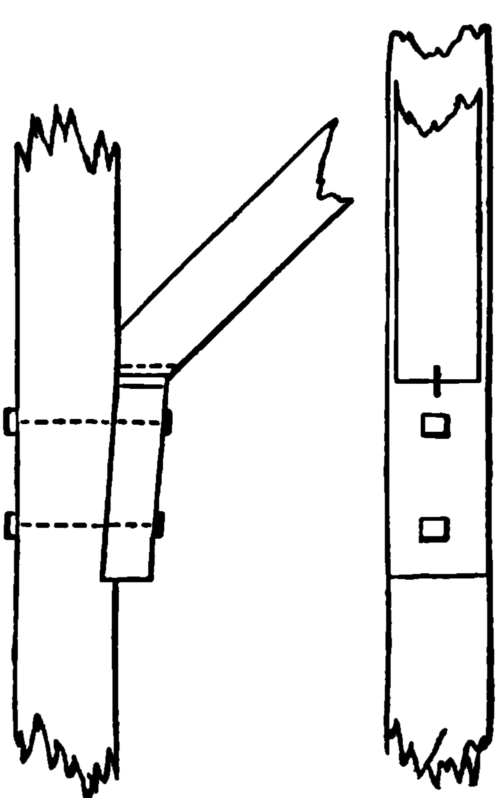


Fig. 18.

structure becomes ricketty, and the strut is generally split by the cross-strain on the bolts. Cases of this kind are most readily treated by running a dowel through both timbers, or by making a vertical saw-cut in each and inserting a strip of boiler-plate.

Struts, Figs. 19, should not be notched at their lower ends into the bearing piles; they should rest on cleats placed with the grain of the timber upright, so that the struts will not be allowed to drop by any shrinkage of the cleats, as might happen if the struts rested on a waling. A strut may be kept in its place by a dowel driven through its lower end, or, still better, by an upright strip of boiler-plate let half into the cleat and half into the strut.



Figs. 19.

In a timber bridge, having continuous floor beams, it is preferable, instead of lapping or scarfing them over the capsills, to cut them of greater length than the spans and to lay the ends side by side, as in Fig. 20, and to bolt or dowel them together. This makes a much stronger combination than scarfing. It confers a great degree of stiffness, and obviates the chance of decay by rain-water getting into and rotting the scarfs.

In the framing of the trusses of bridges the ordinary king-post or queen-post truss is to be avoided. The slightest

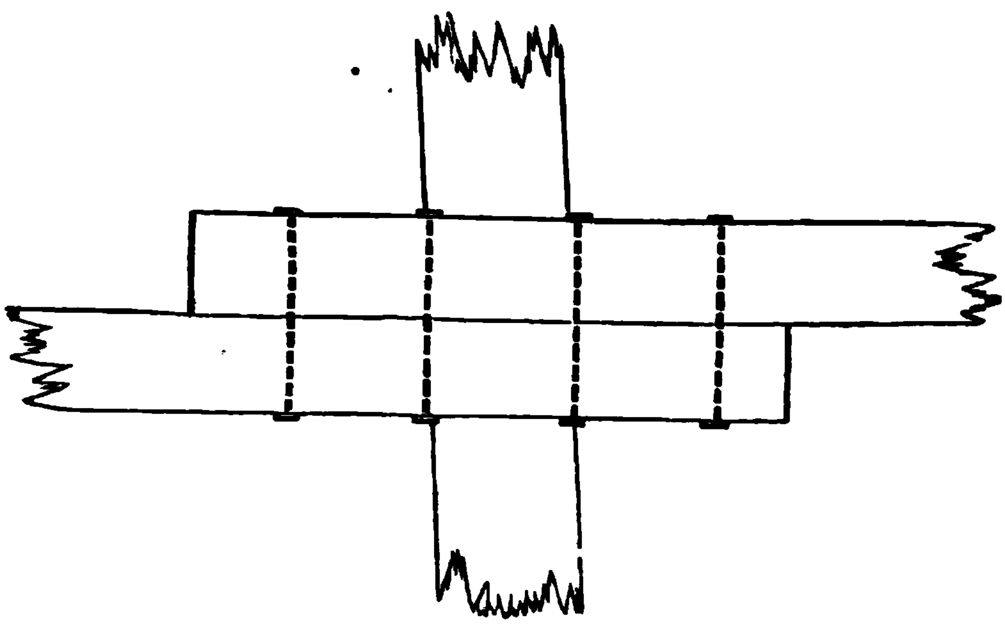


Fig. 20.

shrinkage in the head of the king-post produces very serious deflection of the truss. In such positions cast-iron heads should be used. The feet of the strut should be cut square to the direction of the fibre, as in Fig. 21, and they should rest either in cast-iron shoes or on level blocks of hard-wood, so that shrinkage of the truss may not incur settlement.

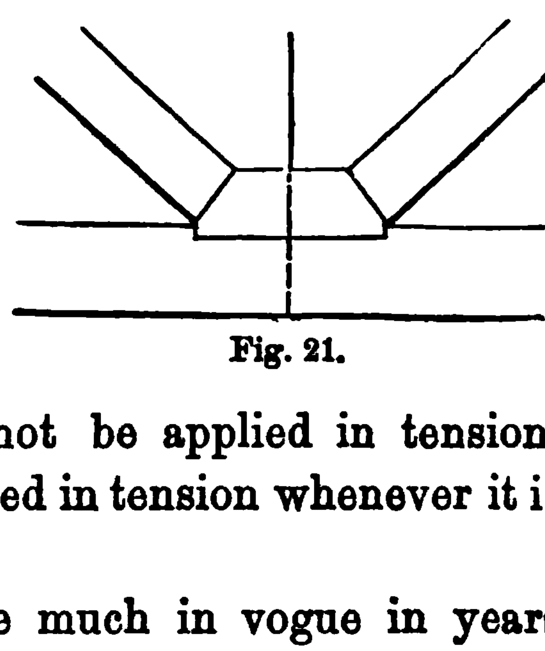


Fig. 21.

In large trusses timber should not be applied in tension. Wrought-iron bars should be applied in tension whenever it is practicable to do so.

Laminated timber arches were much in vogue in years past. They are now generally abandoned. They are difficult of repair.

Timber culverts are a necessity in a timber district, and in other situations in new countries where other material is not available. In the first case, they are made of logs roughly squared, and of sufficient size and weight to keep their position without either bolts or dowels. The roof of the culvert should be formed of stout poles laid across the

road, so that carriage wheels may not fall in between the pieces, as may happen when the roof is constructed of timber laid transversely to the axis of the culvert.

Sawn-timber culverts should be made in lengths, the top, sides, and bottom being framed separately, and put together as in Fig. 22.

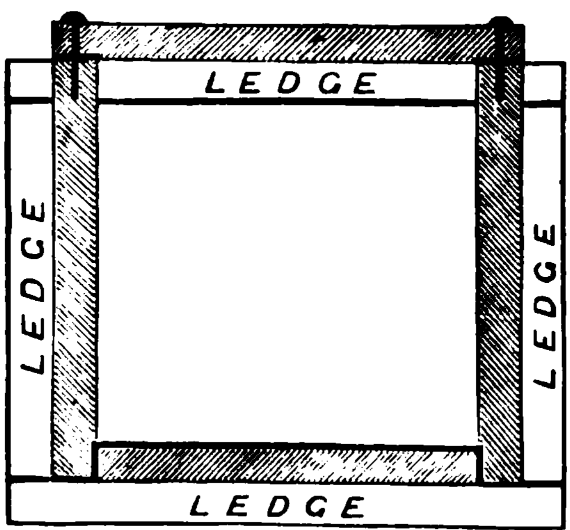


Fig. 22.—Timber Culvert.

Crib-work should be constructed of logs as nearly as possible of the same diameter, say from 8 to 14 inches, and in lengths to be conveniently handled. The lower side of each log is left round, the top is notched to receive the log above it, and all the intersections are securely trenailed. Crib-work executed in this manner, and filled with broken stones, is an excellent substitute for masonry in building breast-walls, crossing gullies, forming embankments or piers in running water, and for like purposes.]

CHAPTER IV.

EQUILIBRIUM OF ABUTMENTS AND WALLS.

We have next to examine the conditions of the stability of piers, abutments, or walls, sustaining some external load or strain, such as the thrust of an arch or the pressure of earth or water. Walls and abutments are usually exposed to two forces—their own weight acting in a vertical direction through their centre of gravity, and the pressure occasioned by the extraneous load which they have to sustain; and upon the magnitude and direction of the resultant of these pressures the stability of the structure depends. They may yield or give way in three different ways: namely, the wall or abutment may separate into two portions, one sliding or slipping upon the other; or it may similarly separate and the upper portion turn over about one or other of its edges; or the material of the wall may be crushed by the pressure exceeding its cohesion. Or, in case the wall or abutment itself is too strong to be broken or crushed, it may still yield in any one of the above ways, by either sliding upon the surface of the ground, or turning over upon one of its lower edges, or from the ground yielding under the pressure. For example, let A B C D, Figs. 23 and 24, represent two walls, each sustaining a pressure acting in the direction G I; let E F be the vertical line passing through the centre of gravity, H the point in which it is intersected by the direction of the pressure; also let H K represent the weight of the wall, and H I the amount of the pressure; then the diagonal H L will

represent their resultant acting in the direction H M. Now, let N O, Fig. 23, be a joint in the masonry of the wall; then (neglecting the adhesion of the cement) if the angle M H F, which the resultant makes with the perpendicular, be greater than the limiting angle of resistance, the upper portion of the wall A B N O will slide upon the lower portion N O C D; and if the adhesion of the cement (being now taken into account) is sufficient to prevent the wall separating at N O, then will the whole wall A B C D slide bodily upon the ground in contact with its base C D; if, however, the angle which the resultant H M makes with the perpendicular to the joints is less than the limited angle of resistance, the wall cannot yield by the sliding of its parts upon each other; and the stability of the wall or abutment will be greatest in this respect when the direction of the resultant H M is perpendicular to all the joints and also to its base C D.

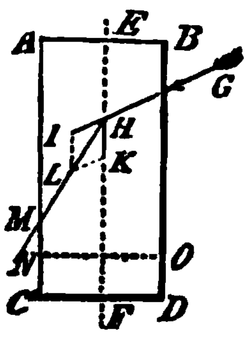


Fig. 23.

If the resultant H M, instead of falling within the base of the wall, cut the side A C, as in Fig. 24, then will the wall separate at the nearest joint N O, and the upper portion will be overthrown, turning upon its edge at N; should, however, the adhesion of the cement be sufficient to prevent the separation of any of the joints, then will the whole wall, A B C D, be thrown over bodily, turning on its lower edge C. The wall, however, cannot be overthrown, so long as the resultant keeps within its substance, and cuts the base C D; and its stability in this respect will be the greatest when the resultant passes through the centre of its base C D.

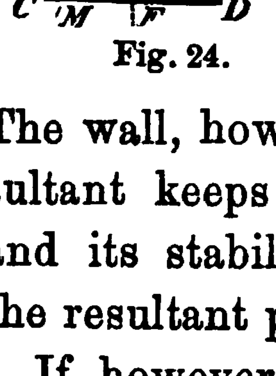


Fig. 24.

If, however, both the foregoing conditions be fulfilled, that is, if the resultant pass through the centre of the base, and its direction be perpendicular to the same, the wall or abutment may still give way by the crushing of its material, or

by the yielding of the ground on which it stands, if the amount of the resultant pressure is greater than they are either of them capable of supporting.

The external pressures to which walls are most frequently exposed are those occasioned by the thrust of arches or the principals of a roof (both of which we have sufficiently explained the method of determining); and also, in the case of retaining walls, the pressure resulting from earth or water sustained by the wall, which latter case we shall next proceed to consider.

Pressure of Earth or of Water against Walls.

When any kind of earth is thrown up into a heap, the sides assume a certain inclination, which is termed the *natural slope*, and is equal to the limiting angle of resistance, or the angle at which a mass of the same earth would commence sliding down its side.

When a mass of earth, supported by a wall, as in Fig. 25, gives way, in consequence of the insufficiency of the wall, it is usually found to separate on some plane D G, the prism of earth B G D sliding down the plane G D, and overthrowing the wall by its pressure against the back B D. Let G D be the inclination at which the earth separates, and let us suppose the mass B G D to be on the point of sliding, or just kept in equilibrium by the resistance of the wall. Now, the two pressures acting upon the mass B G D are its weight acting in the vertical line H I, and the resistance of the wall acting in the direction L I; then, if we represent the former by O I, and the latter by P I, the diagonal N I will be their resultant, and will represent the pressure of the prism B G D upon the plane G D; then, since it is upon the point of sliding down

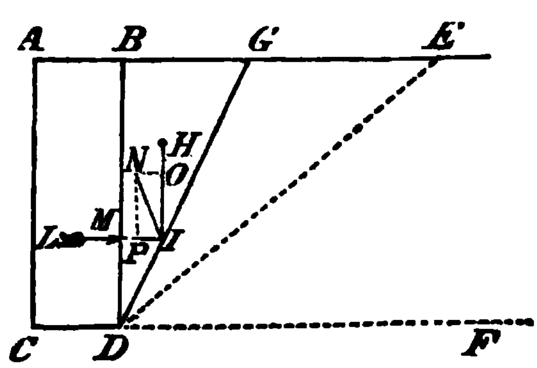


Fig. 25.

this plane, the angle which N I makes with a perpendicular to the surface of the plane D G must be equal to the limiting angle of resistance. Now, the weight of the mass of earth B G D is equal to B D, multiplied by half B G, and by the weight of a cubic foot; therefore, the more G D is inclined, the longer B G will be, and the greater will be the weight of the earth which the wall has to support, and the length of the line O I which represents it; but the more D G becomes inclined the nearer will N I approach to the vertical H I, and therefore the less will be the line P I representing the pressure of the earth against the wall. It must, therefore, follow that there is a certain inclination for the plane D G, which occasions the pressure on the back of the wall to be greater than any other, and this is found to be when the angle B D G is half that which the *natural slope* of the earth D E makes with the vertical, the angle E D F being the limiting angle of resistance.* Now, in this case, it may be shown that the triangle N O I is similar to G B D; and therefore, if B D represent the weight of the mass B G D, B G will represent its pressure against the wall, that is, the weight of the earth is to its pressure against the wall as the height of the wall is to B G; and since the weight of the earth equals the height of the wall, multiplied by half B G, and by the weight of a cubic foot of the earth, *it follows that the* pressure of the earth against the wall *is equal to half the square of* B G, *multiplied by the weight of a cubic foot of the earth.* With the same earth, B G always bears the same proportion to the height of the wall, which proportion for the different kinds of earth is given in the fourth column of the subjoined table, the height of the wall being taken as unity, and the fifth column contains half the square of this fraction, multiplied by the weight of a cubic foot of the earth. In order, then, to determine the pressure produced against a wall by different kinds of soil, we have only to *multiply the square of the height of the wall in feet by*

* Moseley's "Mechanical Principles of Engineering," p. 445.

the number contained in the last column of the subjoined table, and the product will be the pressure in pounds, acting horizontally against the back of the wall at a point (M, Fig. 25) *one-third of the height of the wall above its base.*

Nature of the Earth.	Weight of a cubic foot in pounds.	Limiting angle of resistance = E D F. °	′	Value of B G, the height of the wall being 1.	Constant multiplier.
Fine dry sand	94	30	0	·577	15·666
	119	40	0	·466	12·938
Loose shingle, perfectly dry	106	39	0	·477	12·058
Common earth, perfectly dry and pulverulent	94	43	10	·433	8·815
The same, slightly moistened, or in its natural state	106	54	0	·325	5·595
Earth the most dense and compact	125	55	0	·315	6·213

The numbers obtained by the foregoing rule represent the *active pressure* which the earth exerts against the wall tending to push it over about the point O, and must not be confounded with the *passive resistance* which it would offer to prevent the wall being overthrown in the contrary direction about the point D. In the first case, when the wall is on the point of moving the mass of earth B G D is about moving *down* the inclined plane D G, pushing the wall before it; while in the second case, when the wall is about to move the same mass is on the point of being pushed *up* the incline. Upon this supposition the angle B D G becomes equal to the complement of its former value,* and therefore the resistance calculated for this new value of B G would be much greater than before. The result, however, of mathematical reasoning in this case gives a value for this resistance far greater than it would be safe in practice to calculate upon; because the ground not being incompressible would yield from that

* Moseley's "Mechanical Principles of Engineering," p. 448.

cause, and allow the wall to move along before the amount of resistance which this calculation would show the ground to be capable of producing had been exerted.

In the case of walls supporting water, such as dock walls and quay walls, the resultant of the pressure of the water against the whole surface of the wall is a pressure acting horizontally at a point two-thirds of the depth of the water below its surface, and equal in amount to the square of the whole depth of the water in feet, multiplied by 31·25, the product being the pressure in pounds. The same rule will determine the pressure of water against lock gates, or any other vertical surface. The pressure of water increases with its depth, and is equal at any point to the depth in feet multiplied by 62½ lbs. (the weight of a cubic foot); therefore, to determine the pressure on any surface entirely immersed in water, whatever may be its position, whether vertical, horizontal, or inclined, we have only to multiply the area of the surface in square feet by the depth in feet of its centre of gravity below the surface of the water, and by 62½.

In the case of walls sustaining water, the active resistance and the passive pressure are precisely equal.

CHAPTER V.

METHODS OF FORMING FOUNDATIONS.

THE formation of a firm and secure foundation upon which to build a structure is frequently one of the most difficult operations which the engineer has to perform, and the method adopted must depend upon the peculiar circumstances of the case. When the natural ground is firm, and sufficient to support the weight of the structure to be placed upon it, it is only necessary to make its surface level; when, however, the original surface has a considerable slope, it will not be necessary to bring it all to one level, but it may be cut into a series of level benches or steps.

In most cases, however, the ground is not sufficiently firm to be trusted, and it is found necessary to adopt some artificial means of increasing its resistance. One of the most common methods of doing this is to drive long pieces of timber, termed *piles*, vertically into the ground, until the resistance which they offer is sufficient; when they are all sawn off to the same level, and a platform of timber formed on the top of them, upon which platform the intended structure is built. The piles are usually about 12 inches square, and are pointed at the lower extremity and shod with iron, to enable them to penetrate the ground more easily; they are driven into the ground by the repeated blows of a heavy weight allowed to fall under the influence of gravity on their upper extremity, which should be surrounded with a hoop or ring of iron, to prevent the pile being split by the blows.

Fig. 26 is a section of the wall of the old docks at Hull, supported upon three rows of piles driven three feet apart; each pile is eight inches square, and ten feet in length.

Another method is to throw the base of the structure over a large superficial area, which is sometimes done by spreading large masses of concrete over the ground, and spreading out the wall with footings, or by laying large flat stones over the ground as a foundation course upon which to commence building. When the soft ground is only superficial, and becomes firm at a greater depth, it is usual to excavate the loose strata, and fill up to the level of the bottom of the intended masonry with concrete. When a structure is supported upon piers or detached pillars, the bases of which do not cover a sufficient area to support them without danger of settlement, the weight which they carry may be spread over a much larger surface by turning inverted arches between them, as shown in Fig. 27.

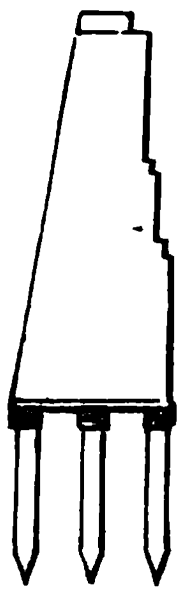


Fig. 26. Piled Foundation.

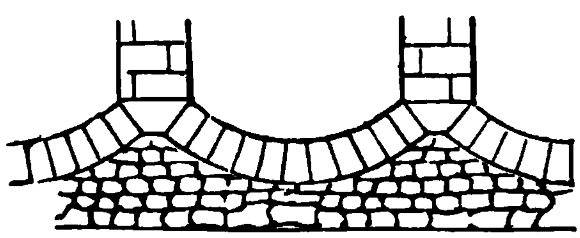


Fig. 27.—Foundation of Inverted Arches.

It sometimes happens that although the ground generally may be firm, in one or two spots it may be loose or soft, and not capable of sustaining the requisite load; in such cases an arch may be turned over the soft place, if not too extensive; in cases, however, where it has been too wide to be spanned by an arch, wells of brickwork have been resorted to, which have been sunk down to the firm ground, and then the structure built upon them. This plan was resorted to by Sir Christopher Wren in building the chancel of St. Paul's Cathedral, under one corner of which a large pit, or *pot-hole*, of loose ground was found.

In conclusion, we must not omit to mention the screw-pile, invented by Mr. Alexander Mitchell, and which has been used with great success for the foundation of light-houses, in situations where from the depth of water or loose nature of the bed, any of the ordinary means would have been totally inefficient. It consists of a large spiral flange or screw of iron, making about one turn and a half, as shown in Fig. 28; it has a square spindle at A, upon which the pile or column A B is fixed; and it is secured in the ground by screwing it down to any depth that may be found requisite, which is easily effected by turning round the pile A B.

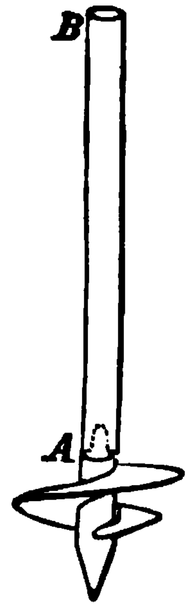


Fig. 28. Screw Pile.

[Foundations for large works of construction are laid either on the natural rock or soil, if sufficiently firm, or on prepared surface where the soil is of unequal density and resistance, or is friable, soft, or subject to disturbance by the action of water or any other external cause. The nature of the soil is ascertained by borings. The most solid bottoms are those which are least liable to compression and lateral movement under a superincumbent load, as unstratified rock; rock in which, if stratified, the seams are horizontal, or are at least strongly cohesive; gravel, dry sand, pure clay, and other compact earths in their natural state. Faults and fissures are frequently met with in rocky formations, occupied by material of less density and resistance than the rock. Such weak places are spanned by an arch, or they are cleared out and occupied by rubble, masonry, or by concrete; or piles are driven into the weaker material, or the area of bearing surface is increased. Beds of rock, with partings of clay between them, are not to be trusted, especially if inclined in direction, as they are liable to slip, and may thus cause serious derangement of the superstructure, if the tendency

be not properly met. In building a bridge, for instance, Fig. 29, on inclined strata of such a nature, over a ravine, whilst the foundations on the one side would be perfectly secure, those on the opposite side would always be liable to disturbance.

Fig. 29.—Foundations on Inclined Strata.

A uniformly weak soil affords a better foundation than a soil of greater strength but of unequal density, since on the former the settlement is uniform, on the latter unequal. The Campanile, or leaning tower, of Pisa, is an instance of a structure on a base of unequal resistance. It is a circular tower, 178 feet high, weighing 11,800 tons, on a base of 60 feet in diameter, equivalent to a pressure of 4 tons per square foot. The soil is of unequal density, becoming weaker as the River Arno is approached. In this direction the tower leans, having settled unequally, and been thrown out of its originally vertical position. That the settlement took place during the progress of the work may be inferred from the presence of the bars of iron introduced in the first and second order to hold the mass together, and the difference of the height of the columns of the fifth order. Almost all the towers of Pisa, as well as the observatory erected in 1755, incline towards the river.

Pure clay, sufficiently beneath the surface to be protected from atmospheric influence, is capable of bearing a pressure of 5 tons per square foot, although the pressure of the Nelson column, in Trafagar Square, London, does not exceed 1·30 tons per square foot. This column rests on clay of great depth and compactness. An excavation 80 feet square and 12 feet deep was made and filled with concrete to a depth of 6 feet. On this base a frustum of a pyramid 48 feet square at the base, and 13 feet high, was built of brickwork, on which the superstructure was built. On a base 60 feet square, which may be taken as the real base of active support, the gross load amounts to 4,665 tons, equivalent to 1·30 tons per square foot, as above stated.

Foundations on gravel or on dry sand, if the stratum be equal in depth to the average breadth of the foundation, may be taken as practically incompressible. The Campanile of Cremona, 395 feet high, standing on pliocene gravel, bears with a pressure of 12 tons per square foot of its base. Water is not likely to be injurious to a foundation of gravel, as it may percolate freely through the material; but to a foundation of sand it is dangerous, and it frequently destroys the character of sand as a natural bearing stratum. A case in point is supplied by the instance of a chimney shaft, 90 feet high, built upon quicksand. The borings showed the existence of sand and water to a depth of 29 feet below the surface. A 1-inch iron rod pitched upright at the bottom of the excavation, which was 16 feet deep, sunk by its own weight 15 feet into the sand. To prepare the sand for the reception of the chimney, it was weighted with concrete to the extent of 10 cwt. per square foot; whilst the superficies of the excavation was fixed upon the supposition that the final maximum load would amount to 1 ton per square foot, the excavation being 22 feet square and 16 feet deep. Concrete was thrown in to a depth of 8 feet, making a gross weight of 210 tons, and it was covered with a layer of 6-inch

Yorkshire flags. Upon this the brickwork was commenced, four courses in cement, 19 feet square. The footings were decreased by half-brick offsets, until the work was carried up to within 6 inches of the surface, where it was reduced to 9 feet 3 inches square. The chimney shaft, 90 feet high, was here commenced, and the settlement was about ¾ inch per day, until a height of 20 feet was reached. It was then increased, when the settlement averaged 1 inch per day for five days; and lastly it was decreased, until the last 15 feet was added, when there was no settlement at all. The total settlement amounted to 16½ inches, the work remaining perfectly upright and without the slightest crack. The total weight of the work, including the filling-in, amounted to 492 tons, on a base of 484 square feet, being at the rate of 1·02 tons per square foot.

The obvious tendency of sand saturated with water to escape laterally, under the pressure of a heavy load, is counteracted by sheet-piling driven well down around the foundation below the base. The tower of the Hamburg waterworks, erected by Mr. W. Lindley, supplies an instance of this kind. The tower rises about 290 feet above the surface of the ground, built of brickwork reposing on a circular mass of concrete 11 feet thick and 56 feet in diameter, founded on quicksand enclosed in sheet-piling driven below the line of saturation of the river Elbe. The gross weight supported amounts to 5,310 tons, being at the rate of fully 2 tons per square foot of base, on the quicksand.

Platforms of timber, or fascines, may be employed upon weak soils to increase the bearing surface or area of resistance, provided they be constantly wet and subject to uniform pressure.

Piles and Pile-driving.

Soft bottoms may be consolidated by driving piles into them, after having been surrounded by sheet-piling, to pre-

vent lateral divergence of the soil. The piles are then sawn off level, the ground between them removed for a depth of two or three feet, the excavation filled with concrete, and covered with planking to form a platform for receiving the superstructure. Occasionally the planking is laid, not on the heads of the piles direct, but on a network of horizontal timber, as in Fig. 30. A pile, 12 inches square, driven 20 feet

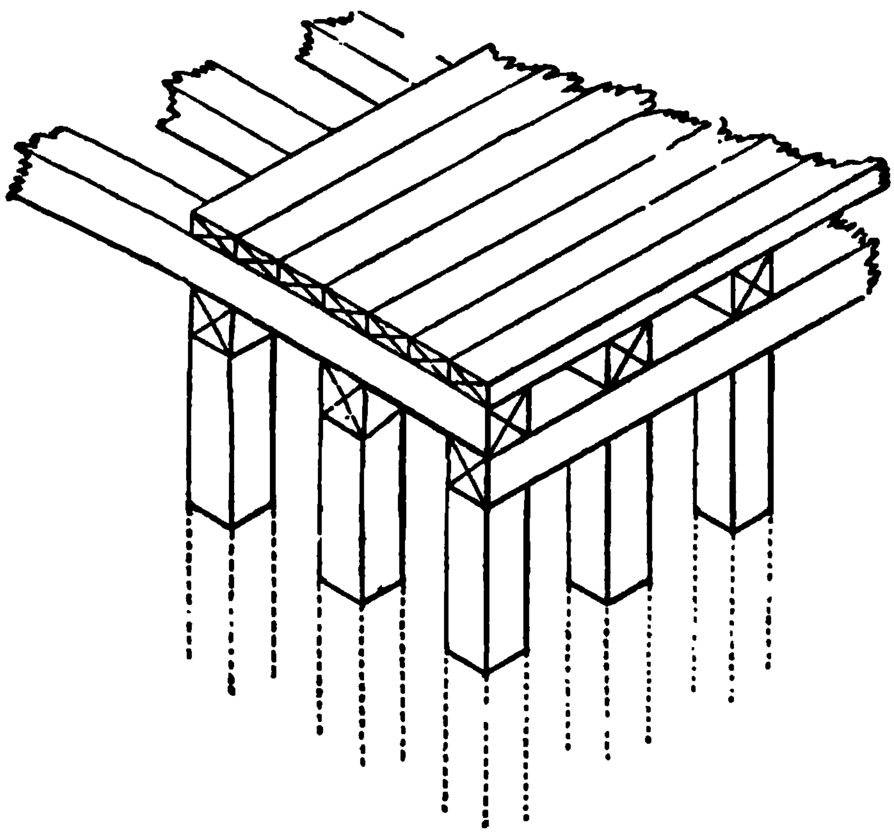


Fig. 30.—Pile Foundation.

into ooze or muddy sand, will not bear more than 9 tons of load. Driven into moderately compact clay, it will bear 12 tons; into hard clay, it will bear 25 tons; and if it reaches to a stratum of compact gravel, as much as 80 tons. From the results of direct experiments made by Mr. R. P. Brereton on the loads up to the breaking point, borne by large fir or pine piles, 12 inches square, of various lengths, the following table has been constructed from plottings by Mr. Stoney :—

Ratio of length to least breadth	10	15	20	25	30	35	40	45	50
Weight that can be borne in tons per square foot of section	120	118	115	100	90	84	80	77	75

This table agrees well with the results of experiments by Mr. Kirkaldy on balks of Riga and Dantzic timber, about 13 inches square, having a length of 20 feet:—

	Total.	Per square foot of section.		By Mr. Stoney's curve.
Riga .	148 tons, or	126 tons	...	116 tons.
Dantzic .	138 „ „	116 „	...	116 „
	Means .	119 „		116 „

Another method of treating soft bottoms is to excavate holes to the depth of the soft ground and refill them with sand, gravel, concrete, or other incompressible material. This system is but little employed in England, but it is much used on the Continent, the method usually followed being to drive down a pile through the soft material, then to withdraw it and fill the hole with sand.

In driving piles for a foundation, there are three special cases:—1. That of a pile driven through a soft stratum to rest on a hard bottom. 2. That of piles driven into ground more or less capable of compression, for the purpose of obtaining support from lateral pressure. 3. That of piles driven into moderately firm ground for the purpose of keeping them fixed in an upright position, like pins in a pincushion.

In the first case the depth of the bearing stratum must be ascertained, and the piles must be of sufficient length. The work done is squeezing rather than hammering until the bearing stratum is reached, when the driving must be conducted with great care to avoid splitting the piles. In the second case, trial piles must be driven to ascertain the depth to which they will go. In the third case, in which a great portion of the pile generally remains above the level of the ground, it is necessary either to use a high engine for driving, or to erect a staging at the level to which the heads of the piles are to be cut off when fully driven.

It is always well to drive with a heavy ram and a low

fall, rather than with a light ram and a high fall. Mr. Dobson* gives the following scale for the weights of rams :—

For piles	10	inches in diameter,	use a	15	cwt. ram.
"	12	"	"	20	"
"	15	"	"	30	"
"	18	"	"	40	"

When a considerable number of piles are to be driven, it is less expensive to use steam power than hand labour.

Hydraulic Foundations.

Hydraulic foundations are such as are laid in rivers, and where else water in motion is to be dealt with. Foundations are laid on natural surfaces when they consist of rock, or on beds of gravel, sand, or stiff clay secured against scour by aprons, sheeting, rubble-stones, or other means of protection. When the foundations are to be laid or pumped dry, the ground is enclosed by dams where the depth of water, if the water be still, is less than 10 feet, or under 3 or 4 feet in running water. A clay puddle embankment, or even one of earth free from stones and roots, forms a sufficient dam. A trench is dug for its foundation, so as to remove loose and porous material from the surface of the ground. The leakage of a dam and the danger of breaches increase rapidly in proportion to the head of water. A solid dam may be made of concrete, but it is expensive to construct and troublesome to remove.

Cofferdams.

In greater depths cofferdams are constructed, taking up less room and being less liable to be water-worn or breached than an earthwork dam. A cofferdam consists essentially of two parallel rows of main piles and sheet piles, enclosing between them a vertical wall of clay puddle. The upper wales of the two rows of piles are tied together by cross

* *Pioneer Engineering*, 1877, page 150.

beams, which support a stage of planking for the workmen. The main piles in one row are at distances of from 5 to 10 feet apart. The ground is excavated between the rows of sheet piles until a sufficiently firm bottom is reached, and the puddle is rammed down in layers. The width of a cofferdam is often as great as the head of water; but if the cofferdam is strutted inside, so that the clay merely acts as a watertight lining, the width need not exceed from 4 to 6 feet. When the height exceeds from 12 to 15 feet or so, three or four parallel rows of sheet piling are driven, thus dividing the thickness of the dam into two, three, or more equal divisions, each of about 6 feet thick. In constructing a cofferdam the first step is to drive guide-piles at short intervals along the line of dam, and to bolt on to them horizontal timbers, or walings, to guide the sheeting piles in their descent. The guide-piles are of whole timbers, the walings generally half-balks. The dam erected for the entrance to St. Katherine's Docks, London, is shown in Figs. 31 and 32.

In cofferdams enclosing a limited area, as, for instance, the site of the pier of a bridge, the required strutting to resist the pressure of water is placed within the dam, across from side to side, the struts being removed as the work proceeds. In constructing dams for a wharf wall, in front, the strutting is differently applied: a series of buttresses, or counterforts, are placed at short intervals, from which dams are strutted, with raking horizontal struts, as exemplified in the cofferdam used in the construction of the river wall of the Houses of Parliament, Figs. 33 and 34.

The cofferdam, Fig. 35, constructed for the works of the Great Grimsby Dock, in the river Humber, is an excellent example of a dam constructed to resist a great head of water, and to withstand storms of great violence. While there was a rise of 25 feet of tide outside, there was inside a depth of excavation 12 feet below low water, made for laying the foundation of the locks. The dock works were commenced

in 1846, from the designs of Mr. Rendel, and completed in

Fig. 31.—Dam, St. Katherine's Docks.

1850. The cofferdam depended solely on its own strength and form of construction for the requisite stability, as there

was nothing in its whole length of 1,500 feet from which it could derive support. In plan, the form of the dam consisted of two circular arcs of 150 feet and 300 feet radius respectively, with a straight return on the west side. The versed sine of the curved portion was 200 feet, or about 1-5th of the

Fig. 32.—Dam, St. Katherine's Dock.

span. The dam consisted of three rows of whole-timber sheet-piling of Baltic yellow pine, from 13 inches to 15 inches square; the outside row battered ½ inch to a foot. The sheeting was all driven between gauge piles, placed 10 feet apart, and the power employed was that of two stationary engines of 30 horse-power, working twelve winding drums,

from which the chains were led to ordinary pile-engines. The last three piles driven in each bay were sawn to a taper

Fig. 33.—Cofferdam, Houses of Parliament.

in opposite directions, so as to wedge the remaining piles of the bay closely together. The piles in the front row ave-

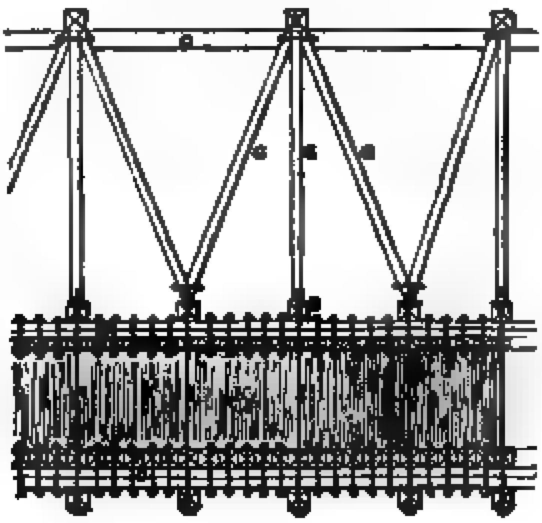


Fig. 34.—Cofferdam; Plan.

raged 53 feet in length, and those of the other rows 45 feet long. The piles were left from 28 feet to 30 feet above

ground, all of them having been driven down sufficiently far to enter a bed of hard clay. The width between the first and second rows of piling was 7 feet; and that between the second and the back rows was 6 feet. The puddle clay occupying these spaces was mixed, for the first 5 feet in height, with one-fourth part of small broken chalkstone, and perfect consolidation was insured by tipping the puddle throughout from earth waggons on the top of the dam. The front and

Fig. 35.—Cofferdam, Great Grimsby Dock.

back rows of piling were secured by five tiers of whole-timber double walings; but in the centre row the three lowest tiers of waling are replaced by bands of wrought iron 6 inches wide by 1 inch thick, keyed together in lengths of 12 feet, and forming a continuous tie on either side of the piling from the two extremities of the dam, and exposing an uninterrupted surface on both sides or faces of the sheet-piling, in order that the puddle might at all times lie closely against

it without leaving any of those voids which are inseparable from the use of ordinary timber walings in such situations, and which serve as channels for any water that may pass along the through bolts. The long bolts broke joint, that is, they were in two lengths, screwed up separately to the central piling. They were 2¼ inches in diameter below, and reduced to 1¾ inches above.

The cofferdam was fortified by a novel system of counterforts or buttresses, each 18 feet in depth and 20 feet in length, consisting of close-driven rows of whole-timber sheet-piling, springing from the back row of the main-pile sheeting, at intervals of 25 feet between centres. This arrangement was found to be completely successful for stiffness and capacity for resisting the pressure of the highest spring-tides. In severe storms the shocks of the waves against the cofferdam scarcely produced any sensible effect.*

Mr. Wm. Cubitt referred to this work as the longest, the strongest, the deepest, and the soundest work of the kind he had ever seen. The cost of the dam amounted to £29 per lineal foot, or to £22 per foot after allowing for the timber drawn.

A form of cofferdam, Fig. 36, buttressed similarly to the cofferdam just noticed, was employed in the construction of the Victoria Embankment on the river Thames, for the Temple Pier. For this pier, irregular in outline, projecting at places upwards of 30 feet beyond the ordinary line of the wall, it was necessary to place the dam so far out as to embrace the greatest projection, and altogether it would have required struts of 57 feet in length. To reduce the length of struts required, and at the same time to strengthen the dam, buttresses were placed at intervals of 20 feet and were 11 feet in width. The bed of the river was dredged out to the clay, into which

* "Description of the Cofferdam at Great Grimsby." By Charles Neate. *Proceedings of the Institution of Civil Engineers*, 1845—50, vol. ix., page 1.

piles 12 inches square were driven, forming a 6-feet space for puddle, with walings and struts 14 inches square. The piles were driven 9 feet into the clay. The rise of spring tides was 18 feet 6 inches, and provision was made for 22 feet rise of water. The dredged-out spaces were filled up on both sides of the dam with a mixture of gravel and clay, which was of great value in increasing the stability of the dam. This dam was 481 feet 6 inches in length. It was found by means of gauges along the upper waling that, at high

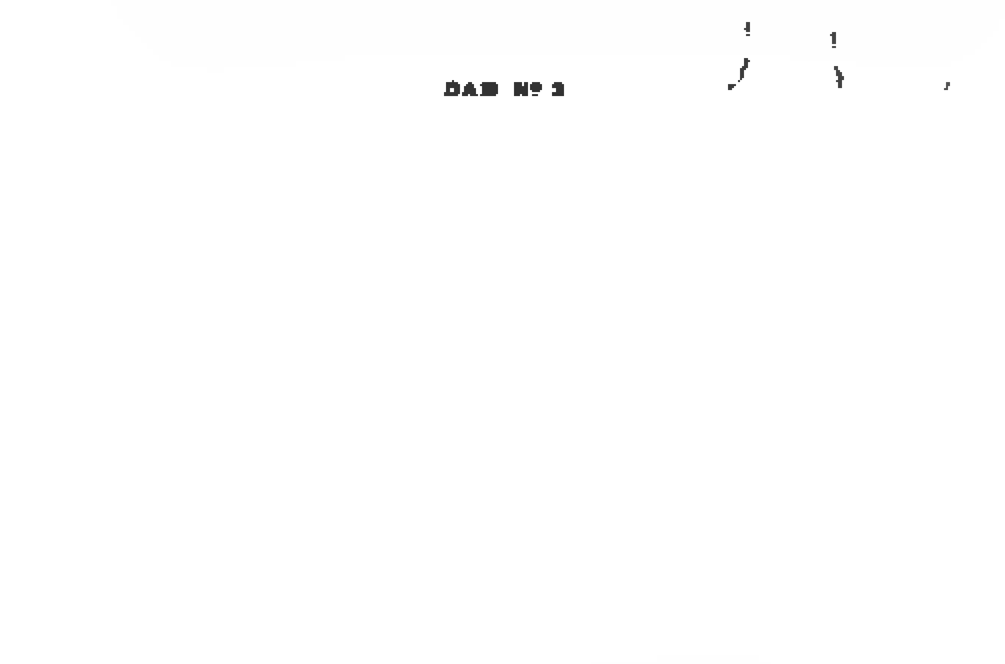


Fig. 36.—Cofferdam, Victoria Embankment.

water, the dam yielded by from $1\frac{3}{4}$ inches to $3\frac{1}{4}$ inches. The puddle was composed of London clay and a sixth part of gravelly loam.

It had originally been proposed by the contractor to construct the dams similar to those, Fig. 33, erected at the houses of Parliament. It was ultimately decided by the engineer, Sir Joseph W. Bazalgette, that the outer face of the dam should not be farther than 15 feet from the foundation trench. Nos. 1 and 2 were therefore constructed according to Fig. 37; made double to exclude the water,

front and back. The remaining dams were constructed as in Fig 38, the inner row of piles being placed so as to

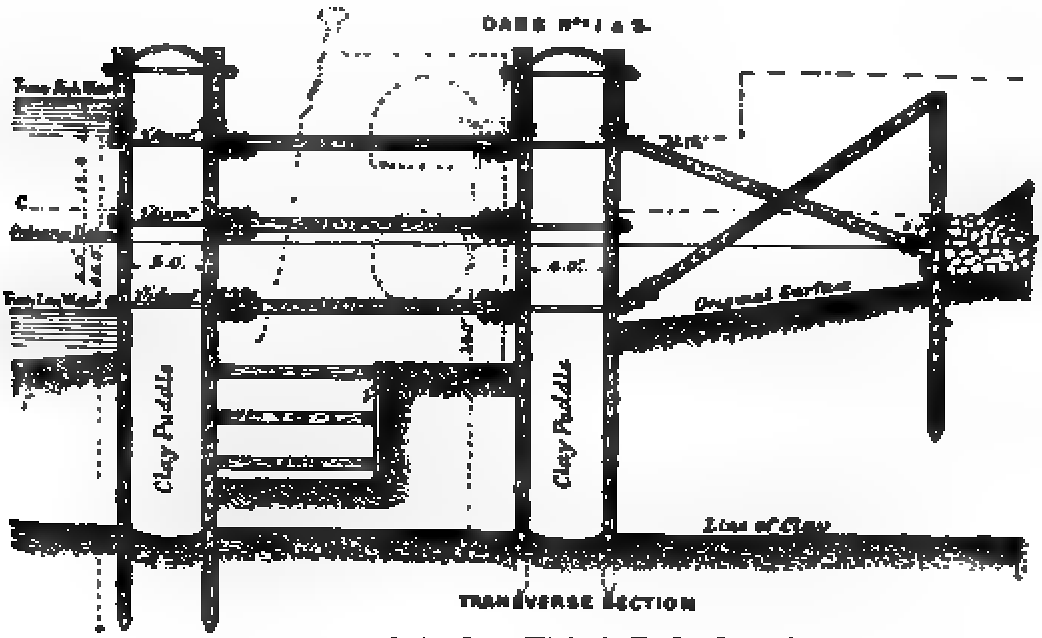


Fig. 37.—Cofferdam, Victoria Embankment.

coincide with the river face of the concrete in the foundation of the wall. Immediately behind each of the back-strut

Fig. 38.—Cofferdam, Victoria Embankment.

piles a mass of rubble-stones was roughly built to add to the resistance and distribute the pressure on the earth-filling.

No. 5 dam yielded by from 1 inch to 4 inches at high water. The yielding was due to the fact that the ground behind the back piles was not sufficiently compact to resist the enormous pressure of the tide.

The quantities of material used in the construction of the dams were as follows:—In dams Nos. 1 and 2, 117 cubic feet of timber, 202 lbs. of iron, 9 cubic yards of puddle, per lineal foot of dam. In the Temple Pier dam, timber 152 cubic feet, iron 285 lbs., puddle 9 cubic yards, per lineal foot. For Nos. 4 and 5 the quantities were nearly the same as for the Temple Pier dam. The cost of the dam was £18 11s. 4d. per lineal foot, and of its removal, £1 4s., together £19 15s. 4d.; nett cost, allowing for value of old material, £17 4s. 10d.

For the construction of other portions of the embankment, wrought-iron caissons were, at the suggestion of Sir Joseph Bazalgette, used instead of the timber dams to effect a saving of cost by using the same caissons two or three times in different parts of the dam. The caissons were constructed of wrought-iron plates, ½ inch and ¾ inch thick, in half oval rings bolted together in pieces, so as to form elliptical sections 12½ feet long by 7 feet wide, and 4½ feet deep. The sections were bolted together vertically, and in all cases rested on a cast-iron section as a base, with a lower cutting edge to penetrate the soil the more easily. The caissons were placed side by side between guide-piles, and were made watertight at the points of contact with felt packing. Each ring weighed 30 cwts., and cost £16 15s. per ton. The base of cast-iron weighed 32 cwts. The caissons were sunk to a depth of 4 feet into the clay by excavating the ground within them, and were weighted with cast-iron blocks of 9 cwts. each. The ground was excavated in three modes: by manual labour, the water being kept down by a chain and bucket pump; by manual labour and pneumatic pressure against the water; and by a telescopic dredger

with endless chain and buckets, the water rising and falling with the tide. The quantity of work done on the three systems respectively was as 60, 45, and 100, and the cost was as 100, 83, and 55. The cost of the dam was £29 10s. per lineal foot, and of its removal 17s. 6d, together £30 7s. 6d.; and, deducting for value of iron used a second time and value of old material, the nett cost was £14 11s. per lineal foot, as against £17 4s. 10d., the nett cost of the timber cofferdam.

The use of single sheet-piling, tongued and grooved, may be resorted to where the structure may be strutted across from side to side, or shored from a solid mass behind it.

Hollow timber frames, without a bottom, and made watertight at the bottom after being lowered, are suitable for building piers of bridges in water from 6 feet to 20 feet deep, on rocky beds, or where there is only a slight layer of silt.

A rubble mound foundation is sometimes used for dams, when any settlement can be repaired by adding fresh material to the top.

A framing not made water-tight may be sunk, inside which concrete is run, and the framing remains as a protection for the concrete and is surrounded by a toe of rubble.

Concrete can be deposited *in situ* for bridge foundations; and though concrete blocks are only employed in sea works, bags of concrete, like those used at Aberdeen by Mr. Dyce Cay, might be sometimes employed instead of rubble-stones for forming the base of piers or for preventing scour.

Cylindrical Foundations.

The piers of bridges in India are, in most instances, supported on wells or hollow cylinders of brickwork. The first length, from 5 to 10 feet high, is placed on a circular wooden framework on the ground. It is then gradually

sunk by a man inside undermining it, and another length is placed on the top. As these operations are generally conducted in the silty or sandy beds of rivers which become dry in summer, there is no running water to contend with; but water percolates into the excavation and then the natives use a "jham," by which they remove the earth from under water.

The use of iron cylinders for foundations, it appears, was first resorted to by Mr. J. B. Redman, at Gravesend. Iron cylinders are preferred, in certain cases, to cylinders of brick, masonry, or concrete, on account of the ease with which they are lowered in deep water on the river bed, and afterwards built up solid with masonry.]

PART I.

INLAND ENGINEERING.

CHAPTER I.

COMMON ROADS.

Determination of Route.

In the laying out of either a canal, common road, or railway, the first and one of the most important points to be considered is the determination of its route or general course. The selection of the best line should be guided by many circumstances, amongst which the following are the most important. The primary object being usually the connection of two distant towns, it is desirable to obtain the most direct and shortest means of communication, which in point of distance would obviously be a straight line. But it is very seldom that a perfectly straight line can be obtained, because there are other requisites equally desirable, which can seldom be attained by taking the most direct route ; these are—as little deviation in the surface of the road or canal from a perfect level as possible (avoiding steep inclines in one case and locks in the other), economy in the construction of the line, the cost of which will be principally affected by the unevenness of the original surface of the country, the nature of the ground, and the number of streams, rivers, roads, &c., required to be crossed by bridges. There is also another

circumstance which frequently makes it desirable to leave the direct and take a somewhat circuitous course, and this is the passing through or near to the intermediate towns lying between the terminal ones, by which the country generally is better served, and an accession of traffic brought to the road or canal.

The course of the best line depending on so many circumstances, it will be easily understood that it requires much care and consideration on the part of the engineer for its selection. In order to obtain the requisite data, or the information upon which to form his judgment, he usually proceeds to a general examination of the district, in which, assisted by some good map showing the physical features of the surface, and accompanied by some person conversant with the country, he ascertains the courses of the valleys and hills, makes general inquiries as to the nature of the strata, the position, population, and trade of the neighbouring towns, and all other points which may affect his selection. He then sketches out one or more lines which appear to him to be most advantageous; these he has carefully surveyed and levelled over, having also cross levels taken by which he may be able to ascertain whether any benefit may be obtained by deviating from the line at first laid down.

Being thus in possession of all the requisite information, he finally determines the course of the line, which is then laid down upon the plan, and also marked on the ground by driving a wooden stake, about 18 inches in length, into the ground, upon the centre of the intended road or canal, at convenient distances, usually a chain (or 66 feet) apart. Very careful levels are then taken over the line thus marked out, every undulation in the surface of the ground being taken notice of; the width of every stream, river, canal, road, &c., is measured, as also the level of its surface and the exact angle (called the angle of skew) which its direction makes with that of the line; it is also necessary,

where the surface of a road crossed by the line will require to be altered, to have levels taken along it for a short distance, so that the exact amount of such alteration may be accurately determined. From these levels a section must be formed representing upon paper the undulations of the ground; in order to render these more easily perceptible, it is usual to *distort* the section by drawing the lengths and heights to different scales. For example, suppose Figure 39 to represent a section of a short line of railway. The irregular line, A B C D E F G, represents the surface of the ground; but, in order to render the undulation in the same more distinct, the horizontal distances are drawn on a scale of 50 chains to the inch, that is, every inch measured along the line I H represents a distance of 50 chains or 3,300 feet on the ground; while the vertical heights are

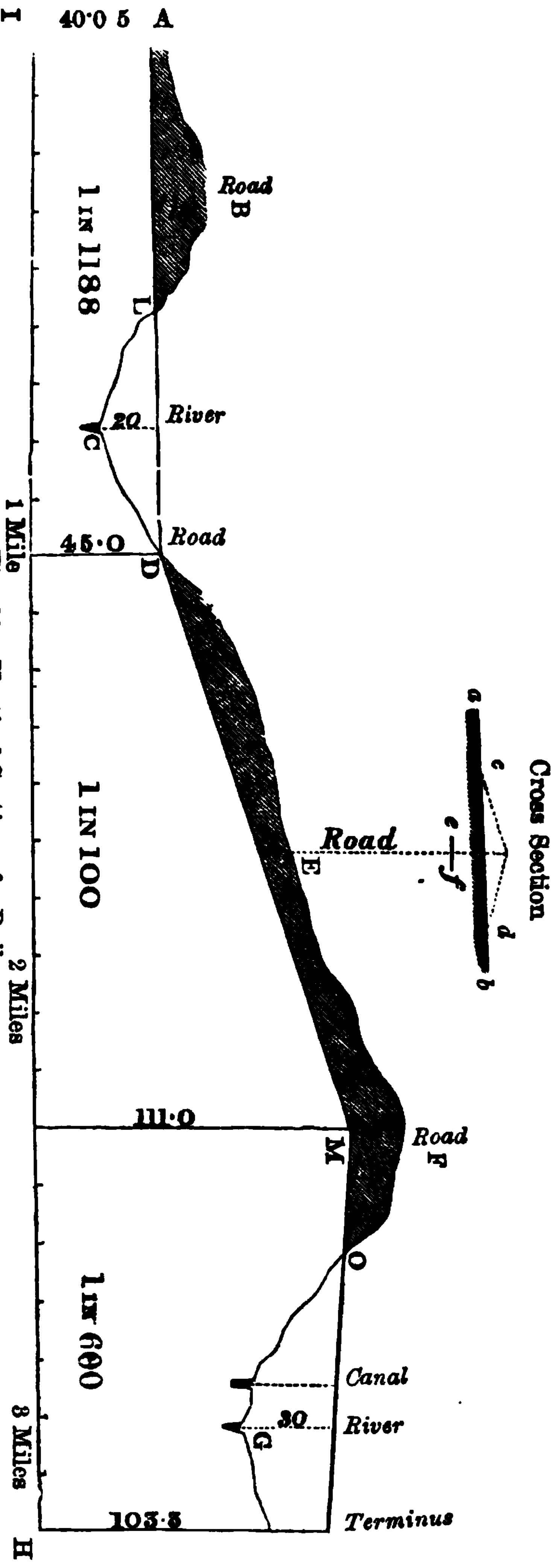


Fig. 39.—Vertical Section of a Railway.

drawn to a scale of 100 feet to the inch, that is, every inch measured in a direction perpendicular to the line I K represents a height of 100 feet. It is usual to refer the levels to some fixed point termed the *Datum*, which in the present instance is taken 45 feet below the surface of the ground at the point A; a line I K called the *datum line*, being then drawn horizontally through the datum, the heights of the ground at any point are always measured from it.

The levels having been taken and the surface of the ground *plotted* or drawn in section, the next step is to determine the levels at which the intended road or railway shall be formed; the latter being the most difficult and requiring the most consideration, will afford us the best example. Now, the principal objects to be borne in mind are—to make the surface of the railway as nearly level as possible, to make the cuttings and embankments balance each other, that is, to make, as nearly as may be, the quantity of ground excavated from the higher parts equal to that required to form the embankment across the more depressed parts, to alter and affect prejudicially the existing roads, &c., as little as practicable, and to keep the cost of the line as low as possible. In the present instance roads are crossed at B, D, E, and F, of which it is desirable that only E should be altered in level; there are also two rivers C and G, both of which require a bridge having a clear headway of at least 17 feet. Now, in order to pass under a road without raising it, the rails must be 18 feet below its surface, and, in order to leave a clear headway of 17 feet at C, the rails must be made 20 feet above that point. If, then, we draw a line at A D fulfilling these conditions, it will represent the surface of the rails, and the railway will be in cutting from A to L, and on an embankment from I to D. The distance A D is 90 chains, or 5,940 feet, and the height of the rails at A 40 feet above the datum, and at D 45 feet above the same, being a rise of 5 feet in a distance of 5,940 feet, or 1 foot in 1,188, which is the inclination of the surface of

the rails, and is technically termed the *gradient*. The next point requiring consideration is the road at F, in order to avoid raising which, the level of the rails must be kept as before 18 feet below its surface; then putting the point M that distance below F, and drawing the line D M, it will represent the surface of the rails, the whole distance being in cutting. The cutting at E being only 10 feet in depth, it will be necessary to raise the road at that point 8 feet, in order to obtain sufficient headway for the railway to pass under it. A cross section must be made similar to that shown in the figure, in which the *whole* line *a b* represents the original surface of the road, the *dotted* line *c d* the proposed surface of the road after being raised 8 feet, the inclination at which it is to be formed being 1 in 20, and the short *thick* line *e f* the level of the rails. The distance from D to M is 10 furlongs, or 6,600 feet, and the rise of the rails 66 feet, or 1 in 100, which is the *gradient* of that portion of the railway. In arranging the level of the line from M to H, we must take care to leave a headway of 17 feet in passing over the river at G, and at first sight this might appear to be the only circumstance to be attended to. If, however, the levels were so arranged as only to leave a headway of 17 feet at the river, the line would terminate with a descending gradient of 1 in 108, which would be very objectionable, because it is always desirable to make a railway approach the terminus on the level, or even with a rising gradient, which latter serves the double purpose of checking the speed of trains coming in, and assists in quickly getting up the speed of those going out. It will, therefore, be advisable to raise the line so as to lessen the rate of inclination, the doing which will be attended with very little expense, because the cutting from D to M will afford all the material required for forming the embankment. If we, therefore, increase the gradient to 1 in 600, the distance from M to H being 7 furlongs or 4,620 feet, the fall in the line will be 7·7 feet, and therefore

its level at the terminus 103·3 feet above the datum; and it will be in cutting from M to O, and on embankment from O to H.

OF THE COURSE, GRADIENT, AND TRANSVERSE SECTION OF THE ROAD.

In determining the course of a road, it is frequently necessary, in order to avoid some obstacle, to change or alter its direction; in such cases, the larger and more regular the curve of the road is made the better, although with common roads it is not necessary to pay so much attention to this point as with railways, and in some instances a very sharp curve or bend may be found necessary.

In arranging the levels of a road (as also a railway or canal) it is very desirable to avoid undulations in its surface, that is, successive inclined planes alternately rising or falling, since much power is required to be expended in going up the hills, while very little will be saved in descending them. When, therefore, the two towns to be connected are nearly on the same level, we should endeavour to make the surface of the road as nearly level as possible; and, when one town stands on a higher level than the other, the connecting road should be formed as nearly as possible with a regular inclination rising from the lower to the higher. This may frequently be partially effected by making the road wind round the side of steep hills, or deep valleys, keeping in each case at the level required; it is, however, very seldom that we can entirely attain this desirable condition of the surface of the road.

When, however, undulations in the surface of the road are unavoidable, we should endeavour to make them as slight as possible, the limit (except in very urgent cases) being that inclination at which a carriage once set in motion upon the road would continue to descend by the action of gravity alone, because, if the hill is steeper than this, the carriage

would have its motion accelerated in descending, and would press upon the horses, urging them forward beyond a safe speed. This limit is attained when the inclination of the road is made equal to the limiting angle of resistance for the materials composing its surface,* and therefore varies with the nature of the road, depending for its value upon the force required to move a given load upon it. The following table exhibits the force required to move a load of a ton on each of the roads described, as also the limiting angle of resistance and the greatest inclination which ought to be given to the road:—

Description of the road.	Force in lbs. required to move a ton.	Limiting angle of resistance.	Greatest inclination which should be given to the road.
Well-laid pavement	33	0° 50′	1 in 68
Broken stone surface, on a bottom of rough pavement or concrete	46	1° 11′	1 in 49
Broken stone surface, laid on an old flint road.	65	1° 40′	1 in 34
Gravel road	147	3° 45′	1 in 45

In arranging the cross section of a road, the width must depend upon the locality and the amount of traffic; for roads much frequented between large towns the width should not be less than 30 feet, with one footpath of about 6 feet in width, and on approaching the immediate neighbourhood of the city it may be increased to 45 or 50 feet, with two footpaths, each of 6 feet. The form of its cross section should be rounding, in order that rain falling upon it may readily drain off and not remain in puddles, which would

* The expression "limiting angle of resistance" is not used here exactly in its ordinary sense, but means the angle at which a carriage once set in motion would continue to *roll* down the incline.

soak through and soften the foundation of the round. The form most usually adopted is that of a flat ellipse, but this is not so good as the segment of a circle, or, better still, two tangents joined by a segment, the ellipse being too flat in the centre of the road and giving too great an inclination at its sides. With a width of 30 feet, the crown of the road should not be more than 6 inches above the sides, and in most cases it would be better not to make it more than 4 inches higher. The surface of roads should always be as much exposed to the free action of the sun and wind, by which rain falling upon it is speedily evaporated and its surface is maintained dry; for which reason high fences or hedges by the sides of roads are objectionable, as are also trees standing by the roadside, which not only impede the sun and wind, but also injure the road by the drippings of rain falling from their leaves. Ditches should be formed on each side of the road to catch the water draining from its surface; and on the side on which is the footpath small

Fig. 40.—Cross Section of a Common Road.

drains should be formed under the same, to lead the water from the gutter on that side into the ditch. Figure 40 exhibits a road 30 feet in width, with one footpath 6 feet wide, having its cross section of the form recommended above, and with side ditches.

However efficiently the surface of a road may be drained by preserving its cross section of the proper form and free from depressions and ruts, and although freely exposed to the action of the sun and wind, unless the superficial coating of the road is very compact, some portion of the rain falling upon it will soak through, and find its way to the founda-

tion upon which the road is formed. It is therefore customary in good roads, in order to remove any water which may thus find its way to the substratum of the road, to form open tile drains across the road at certain intervals, depending upon circumstances, but usually about 60 yards apart, and having a slight inclination from the centre of the road into the ditches on each side. When the road is level, these transverse drains should run straight across it at right angles to its direction; but, when it is inclined, the drains should be formed as shown on the plan, Fig. 41, making an

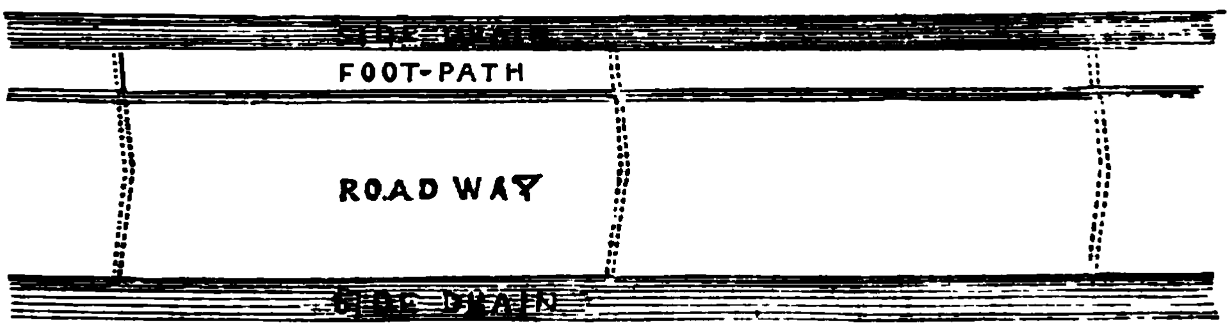


Fig. 41.—Drainage of Road.

angle in the centre of the road, from which point they run straight each way into the side ditch, slightly inclined in the direction in which the road falls.

Of the different kinds of Roads, and the Materials employed in their Construction.

In the construction of any kind of road, the point requiring the first care is to form a good and sufficient foundation; by properly attending to which, although the cost of its formation may be somewhat increased, any additional outlay on that account will be more than repaid by the saving which will result in the expense of repairing the road. In the general practice, the formation of a good foundation is seldom sufficiently attended to, the principal care being usually bestowed upon the superficial coating; if, however, the foundation of the road is deficient, no care or expense bestowed upon the covering will render the road durable.

It should be borne in mind that the substratum is really the working road which has to support the weight of the passing traffic, and that the office of the covering is simply to protect the actual road beneath it from wear.

Roads may be divided into two kinds, those which have their surface protected by paving, whether of stone, wood, &c., and those whose surface is formed by a covering of broken stones, or *macadamised*.* The latter method derives its name from the gentleman, Mr. Macadam, by whom they were first brought into notice.

In forming a macadamised road, if the ground is firm and dry, the only preparation required is to bring its surface to a true level; should it however be at all wet, or of a marshy character, the portion upon which the road is to be formed should be first carefully and thoroughly drained, which may usually be most effectually done by cutting deep drains running parallel to the intended course of the road on either side of it, and, if it is found necessary, forming cross drains between them having a fall each way. The ground having been thus drained, a covering of turf or of brushwood, the latter not less than 6 inches in thickness when compressed, should be laid over the surface of the soft ground, and upon this should be spread a covering of 3 or 4 inches of clean gravel, the upper surface of which should be level. The foundation of the road should now be formed, by laying a kind of rough pavement as shown in the section, Fig. 48, consisting of rough stones of any kind of stone that can be most readily procured, laid carefully by hand with their broadest faces on the ground. These stones should be not less than 7 inches in depth in the centre of the road,

* The term macadamised roads should strictly be applied only to such roads as are formed entirely of broken stones without any rough pavement for their foundation; but of late years it has been found convenient to apply the term to all roads composed of and repaired with broken stones.

gradually diminishing to 3 inches in depth at the sides, and the interstices between them should be carefully filled with stone chippings, so that the upper surface when finished may form a regular curve with a convexity of about 4 inches. The material for forming the surface of the road should then be laid on, forming a uniform coat 6 inches in thickness. For the centre portion of the road care should be taken to select a stone which is hard and not friable; granite, whinstone, and the harder limestones are the best suited for this purpose; and they should be broken into angular fragments, the largest of which should be capable of being passed through a ring 2½ inches in diameter. For the sides of the road well-cleansed strong gravel may be used. A good binding of clean gravel perfectly free from earth or clay, about 2 inches in depth, should then be laid over the entire surface of the road. It is better to put only 4 inches of the broken stone at first, and, after this has become consolidated by the traffic, then to lay on the remaining 2 inches, care being taken, however, to fill up any ruts which may have been formed.

As much care and attention are required for the economical repair of roads as for their first construction. Particular care should be taken that the side ditches and drains are kept clear and free from any obstruction; ruts, hollows, and inequalities in the road should be filled up the moment they appear, the best time for doing which is after wet weather, when they are not only more readily seen, but, the road being then soft, the new material works in without being crushed or ground to powder, for which reason the proper time for the general repair of roads is about April and October. Nothing tends more to the preservation of a road than keeping its surface clean and free from mud, which should be continually scraped off and never allowed to accumulate.

In constructing paved roads, the same care should be

taken to secure a good foundation as in forming macadamised roads. The most perfect foundation for pavement is a bed of concrete, the thickness of which must depend upon the nature of the ground beneath it, but should in no case be less than 6 inches, and its upper surface should be formed with a regular convexity similar to that intended to be given to the road. Two different kinds of materials are used for paving roads, viz. stone and wood; of the former, the stone most generally employed is granite, and the best description of pavement consists of narrow stones not more than 4 inches in thickness and about 9 inches in depth placed edgewise. The stones should be beaten into their places by a heavy wooden beetle and grouted with a thin lime grouting, after which a covering of fine clean gravel, about 1½ inch in thickness, should be evenly spread over its surface. Wooden pavement has been successfully employed for several years in Russia, and has lately been introduced into England, and is far superior to stone pavement as regards the comfort both to passengers and residents, arising from the evenness of its surface and the absence of noise. Many different kinds have been tried, the objects sought for being to prevent irregular settlement of the blocks and to remove the slipperiness of its surface; the former of these objects has not yet been, nor will it ever be, attained while the unyielding quality is sought to be obtained from the pavement itself, and so little attention is paid to the formation of a firm foundation. The timber should merely be regarded as a durable and elastic covering to protect the solid and well-formed road which ought to be first constructed under it.

CHAPTER II.

CONSTRUCTION OF MODERN MACADAM ROADS.

[THE levels, gauges, plummet-rules, and prong-shovels formerly employed in the setting-out and construction of roads are not now used. "Boning-rods" are used for fixing the inclination of the surface, longitudinally and transversely, by the eye.

In the construction of metropolitan roads of the first class, when the ground has been excavated and levelled, it should be rolled when it consists of clay. A bottoming, or bed, 12 inches thick, of "hard-core" is laid on the ground; it may consist of brick rubbish, clinker, old broken concrete, broken stone or shivers, or any other hard material in pieces. The bed is rolled down to a thickness of 9 inches, and any loose or hollow places are made up to the level. Next comes a layer of Thames ballast, 5 inches thick, rolled solidly to a thickness of 3 inches. The ballast serves to fill up vacancies in the bottoming, and, being less costly, saves so much of the cost for broken granite. Broken granite, or macadam, is laid upon the prepared surface of the ballast, in two successive layers, 3 inches thick, rolled successively, to a combined thickness of 4 inches; a layer of sharp sand, ½ inch or ¾ inch thick, should be scattered over the second layer and rolled into it, with plenty of water. But it is better to add the sand and the water as the second layer of granite is laid, and to roll them well together.

Second-class metropolitan roads are usually constructed of

a hard core of brick rubbish or other material, about 9 inches thick, and a layer of broken granite or of flints, about 4 inches thick. The material is not, in general, submitted to rolling other than the action of the traffic.

In transverse section, the contour of the road is a segment of a circle. All roads and streets are or should be circular in section. The general practice is in this respect at variance with the practice recommended by Mr. Law, page 70, when he recommends that the section of the surface should be formed of two inclined straight lines, joined by a flat curve at the middle, forming a species of ridge. The advantage of the circular section consists in the fullness given to the "shoulders" of the road, which lie in the lines of traffic on each side of the centre-line of the road, and thus present a full wearing surface.

Estimates have been formed of the relative tear and wear of roads due to the action of the shoes of horses and the action of wheels. Mr. Telford considered that the tearing up of a well-made road by horses' feet was much more injurious to the road than the rolling pressure of wheels. Sir John Macneil was of the same opinion; and, according to an estimate formed by him, it appears that, for "the generality of roads," the wear of and injury to roads may be apportioned as follows for fast-coach traffic: Atmospheric changes, 20 per cent.; wheels, 20 per cent.; and horses' feet that draw the vehicles, 60 per cent. For waggon traffic, the second and third causes of wear were adjusted in the proportions of 35 per cent. and 45 per cent.

From these and other considerations, it is obvious that the wear of macadam roads must necessarily be much greater than that of paved roads or streets. Mr. Mitchell's remarkable analysis of the material of a macadam road places this conclusion in a clear light.

A cubic yard of broken stone metal, of an ordinary size—2 inches or 2½ inches cube—when screened and beaten down

in regular layers 6 inches thick, contains, according to Mr. Mitchell, 11 cubic feet of interspaces, as tested by filling up the metal with a liquid. Herr E. Bokeberg, of Hanover, who made many very careful experiments on the proportion of vacuity to solid material, found that in a loosely heaped cubic yard of broken stones void space amounted to one-half of the total volume. A large portion of the vacant space becomes filled with mud, which forms the cementing matter, ground from the metal, the primitive mass of broken stone being crushed into every variety of form down to the finest sand. Mr. Mitchell gives the result of an analysis of a portion of the crust—2¼ cubic feet—of the macadamised road in the Mall, St. James's Park, which was taken up for examination. One cubic yard contained:—

	Cubic feet.		Per Cent.
Mud	11·00	or	41
Sand, with pebbles, not exceeding 3/16 inch thick	2·40	or	9
Stones, from 3/16 inch to ½ inch	6·56	or	24
Stones, from ½ inch to 1 inch	4·48	or	16½
Stones, from 1 inch to 2¼ inches . . .	2·56	or	9½
Total volume, 1 cubic yard	27·00	or	100

From this analysis it appears that less than 9½ per cent. of the original stone remained unground, whilst 40 per cent. of it was reduced to sand. These proportions, taken as they stand, are too favourable for the duration of the stone in that instance, for no doubt the sample was a sample of the remains of stone, much of which must have been swept or washed off out of sight. Mr. Burt, then, cannot be far amiss when he estimates that one-third of the loose road material used in London is literally wasted by being ground up under the traffic before the consolidation of the surface is effected.

The logical inference is, that a macadamised road is not properly fit for traffic unless it is condensed, consolidated, and reduced to a hard and regular surface by suitable appli-

ances. The road-roller has long been successfully employed in France and Germany for this purpose. A well-rolled road covering contains at least from 70 to 80 per cent. of mass of stone, leaving only from 20 to 30 per cent. of intermediate space, most of which should be filled, especially at the top, with clean sand. In a cubic yard, the spaces would amount to from 5½ to 8 cubic feet, which would prove that the spaces are reduced by rolling to nearly one-half of the amount when the new metal is not rolled. It is scarcely necessary to insist on the increase of durability, and the clear gain in economy of maintenance of the road, by efficient rolling, which, it seems, was first applied in 1830, although but imperfectly appreciated in England until about the year 1843, when the first-published recommendation in the English language of horse road-rolling, as a measure of economy, was issued by Sir John Burgoyne, in his paper "On Rolling new-made Roads." Steam road-rollers have been employed in Paris since about the year 1864, and were brought into use in England a few years later. Messrs. Aveling and Porter, who have had much experience in the manufacture of steam-rollers, have constructed them of two classes, each on four rollers, weighing 15 tons and 20 tons. They are said to be capable of rolling 2,000 square yards of new macadam per day, at a cost averaging 15s., making an average of 10 or 12 square yards rolled for 1d.

Mr. G. F. Deacon states that at Liverpool, under a 15-ton steam-roller, preceded by a watering-cart, 1,200 yards of trap-rock macadam, without blinding, can only be moderately consolidated by twenty-seven hours continuous rolling. If blinded with hard-rock chippings from a stone-breaker's, the same area may be moderately consolidated by the same roller in eighteen hours. If blinded with siliceous gravel from ¾ inch to the size of a pin's head, mixed with about one-fourth part of macadam sweepings obtained in wet weather, the area may be thoroughly consolidated in nine

hours. Macadam, he adds, laid according to the last method, wears better than that laid by the second, and that laid by the second much better than that laid by the first.

Messrs. Aveling and Porter have since designed a lighter roller of 7½ tons weight, arranged like an ordinary traction engine, running on rollers in place of wheels.

A macadamised road, constructed in the best manner, with metal laid nine inches deep, cost, in 1877, at London prices, 6s. 3d. per square yard. The cost for maintenance of several principal macadamised thoroughfares in London, exclusive of watering, has been stated, by various competent authorities, as follows:—

	Year.	Annual cost per sq. yard. s.	d.
Parliament Street	1856	2	4
Ditto	1869	3	3
Bridge Street, Westminster	1856, 1869	3	6½
Great George Street, Westminster	1856	0	6½
Westminster Bridge (Old)	1854	2	0
Piccadilly	1834-63	2	5
Ditto	1870	3s. 6d. to 4	0
Ditto	1879	4s. to 6	0
Regent Street	1876	3	7
Cranbourne Street and north side of Leicester Square	1870	2	0
Park Lane	1879	3	6
Knightsbridge	1879	3	6
Grosvenor Place	1879	3	0
Buckingham Palace Road	1879	3	0

In the Hackney district of the metropolis, according to Mr. Lovegrove, the statistics of twenty-one different streets showed that the cost, including granite, rolling, watering, labour, and covering material, varied from 9¾d. in High Street, Homerton, to 1s. 9¾d. in Bradbury Street, per square yard per year. The coating of granite laid on varied from a minimum of 1 inch to a maximum of 3 inches in thickness.*

* *Proceedings of the Institution of Civil Engineers*, vol. lviii. p. 62.

For the maintenance of the suburban highways of the metropolis, the cost for maintenance averaged 7¼d. per square yard in 1855. In Birmingham, about the same period, the cost averaged 4d. per square yard, with 2d. extra for watering and cleansing. In Derby, in 1876, the cost for maintenance averaged 11d., cleansing 4¼d., and the total cost was 1s. 3¼d. per square yard per year. In Great Howard Street, Liverpool, the nett cost for maintenance amounts to 3s. 6d. per square yard per year.

For macadam, the hardest stones—as Guernsey granite and Penmaenmaur greywacké—are the most suitable. Their slippery qualities, objectionable in pavement, are of no moment in macadam, whilst their hardness and toughness are valuable qualities. In India, the stones used for macadam are granite, trap, and the hard limestones and sandstones. Laterite, which is a hard sandstone, is very much used in the Madras roads; but it is comparatively soft, and does not bear much traffic. Kunkur is the material chiefly used in India; it is a peculiar formation of oolitic limestone, found generally in the form of nodules, sometimes in masses a little below the surface of the earth. It makes an excellent road, but it requires constant repair if the traffic is heavy.

Asphalt Macadam.

Asphalt macadam, a bituminous concrete, has been successfully practised in Liverpool. It is absolutely impervious to moisture, and has been laid at a cost of 3s. 6d. per square yard, six inches deep; or, including general charges, 3s. 9d. per square yard. The asphalt—a mixture of pitch and dead oil—is poured hot into the stratum of macadam, sinking to the bottom and filling the smallest crevices. On the surface, while the asphalt is still warm, is laid a thin stratum of small broken stone, which is thoroughly rolled into the

asphalt, and fills up the upper spaces between the larger stones. As, in the process of rolling, the soft asphalt rises between the smaller stones, still smaller riddlings are scattered over it, until a perfectly true and uniform surface is obtained.]

CHAPTER III.

ROADS IN FRANCE.

[IN France the causeway, previously to 1775, was generally 18 feet wide, with a depth of 18 inches at the middle and 12 inches at the sides. Stones were laid flat, by hand, in two or more layers, on the bottom of the excavation. On this foundation a layer of small stones was placed and beaten down, and the surface of the road was formed and completed with a finishing coat of stones broken smaller than those immediately beneath. As the roads were, down to the year 1764, maintained by statute labour (*la corvée*), with which the reparations could only be conducted in the spring and the autumn of each year, it was necessary to make the thickness of the roads as much as 18 inches, that they might endure during the intervals between repairs. With less depth, they would have been cut through and totally destroyed by the deep ruts which were formed in six months. The suppression of statute labour in 1764 was the occasion of a reformation in the design of causeways, whereby the depth was reduced to such dimensions as were simply strong enough for resisting the weight of the heaviest vehicles. The depth was reduced to a uniform dimension of 9 or 10 inches from side to side, and the cost was diminished more than one-half. Writing in 1775, M. Trésaguet, engineer-in-chief of the generality of Limoges, stated that roads constructed on the improved plan lasted for ten years, under a system of constant maintenance, and that they were in as good con-

dition as when first constructed. The section of these roads, as elaborated by M. Trésaguet, is shown in Fig. 42. The form of the bottom is a parallel to the surface at a depth of 10 inches. Large boulder-stones are laid at each side. The first bed consisted of rubble-stones laid compactly edgewise, and beaten to an even surface. A second bed, of smaller stones, was laid by hand upon the first bed. Finally, the

Fig. 42.—Old Roads in France.

finishing layer of small broken stones, broken by hand to the size of walnuts, was spread with a shovel. Great care was taken in the selection of stone of the hardest quality for the upper surface. The rise of the causeway was 6 inches in the width—18 feet—or 1 in 36. This system was generally adopted by French engineers in the beginning of the present century, although on soft ground they placed a layer of flat stones on their sides under the rubble-work. In this case the thickness was brought up to 20 inches. The rise of the causeway was as much as 1 in 24, and often equal to 1 in 20.

But, though the design was good, the maintenance was bad. Large and unbroken stones were thrown into the holes and ruts, and neither mud nor dust was removed. About the year 1820, the system of Mr. Macadam attracted some attention in France, and the peculiar virtue of angular broken stone in closing and consolidating the surface was recognised. About the year 1830, it is said, the system of macadam was officially adopted in France for the construction of roads; and M. Dumas, engineer-in-chief of the Ponts et Chaussées, writing in 1843,* stated that the system of macadam was generally adopted in France, and that the

* *Annales des Ponts et Chaussées*, 1843, tome 5, p. 348.

roads were maintained, by continuous and watchful attention in cleansing and repairing them, in good condition, realising his motto, "The maximum of beauty." But the employment of rollers for the preliminary consolidation and finishing of the road has been an essential feature in their construction and their maintenance; for it has long been held in France that a road unrolled is only half finished. It appears, according to Mr. F. A. Paget, that the horse-roller was introduced in France in 1833. At all events, in 1834, M. Poloncean, struck by the viciousness of the mode of aggregating or rolling the material of the road by the action of wheels, proposed, in the first place, to consolidate the bottom by means of a 6-ton roller, and to roll the material in successive layers consecutively, and thus to complete in a few hours what might, in the ordinary course of wheel-rolling, require many months to perform.

The width of the old roads of France was excessive, amounting sometimes to nearly 80 feet, having the 18 feet pavement in the middle, as already described. They are now made of widths of from 8 to 14 metres, or from 26 feet to 46 feet. Type-sections of French and Belgian roads are shown in Figs. 43 and 44. The 6-ton roller already mentioned has a diameter of from 6 feet to 6½ feet, and is 5 feet wide; weighing, empty, 3 tons, and, full, 6 tons. The maximum weight, when loaded, should be from 8 to 10 tons. These weights give a pressure varying from 112 lbs. to 370 lbs. per inch of width, the sufficiency of which has been proved by experience. The empty roller is first used, then the full roller, and lastly the weighted roller. Sand or other binding, with water, is thrown on the surface at intervals. The material binds most speedily when the thickness is 4 or 5 inches. The surface is kept up by the use of a stamper or rammer, weighing from 15 lbs. to 20 lbs., which is 8 inches in diameter at the base, shod with iron. A road thus constructed is superior to a road 8 or 10 inches

in thickness, consolidated by wheels, and with successive additions of material. The flanks or bermes of the roads (*accôtements*), consisting of the natural ground, are fit for

Fig. 43.—French Roads.

Fig. 44.—Belgian Roads.

traffic during the greater part of the year. "All these proceedings," says M. Dumas, "have for their basis the principle of the *maximum of beauty*."]

CHAPTER IV.

STONE PAVEMENTS.

In the City of London, granite sets of comparatively large dimensions were at first used—from 6 to 8 inches wide on the surface, by from 10 to 20 inches long, with a depth of 9 inches. As originally laid, they were merely laid in rows on the subsoil, and after the usual process of grouting and ramming, the street was thrown open for the traffic which was expected to perform the last duty of the paviour, and to settle each stone upon its bed. The large wooden rammer of 84 lbs. weight was insufficient for the purpose of enabling the pavement to resist without further movement the percussion of heavily-weighted wheels. In 1850, and probably for some time previously, it had become the general practice to make a good substratum of "hard core," consisting of shivers, brick rubbish, clinkers, or other hard material, usually laid to a depth of from 9 inches to 12 inches, though 15 inches of depth has been laid in the principal streets. Upon the hard core was laid a stratum of sand, into which the stone sets were bedded. Fig. 45 shows a section of King William Street as originally paved.

The long continuance of the system of paving with large blocks resulted from the experience of their great durability and economy in first cost. But they did not afford sufficient foothold for horse traffic. Granite sets of less width were subsequently laid; they were 5 inches and 4 inches in width; and finally sets of only 3 inches in width were laid. The

3-inch sets, with a depth of nine inches, although they were considered to be the least durable and the greatest in first cost, proved to be by far the safest for paving, and they gave a greater degree of satisfaction than the wider sets. The merit of their introduction is due to Mr. Walker, under whose direction Blackfriars Bridge was, in 1840, paved with 3-inch granite sets. Regarding the question in all its bearings, Colonel Haywood, the engineer and surveyor to the Commissioners of Sewers of the City of London, concluded that the 3-inch granite sets, being safest, as giving the best foothold, were the best for large towns of great traffic.

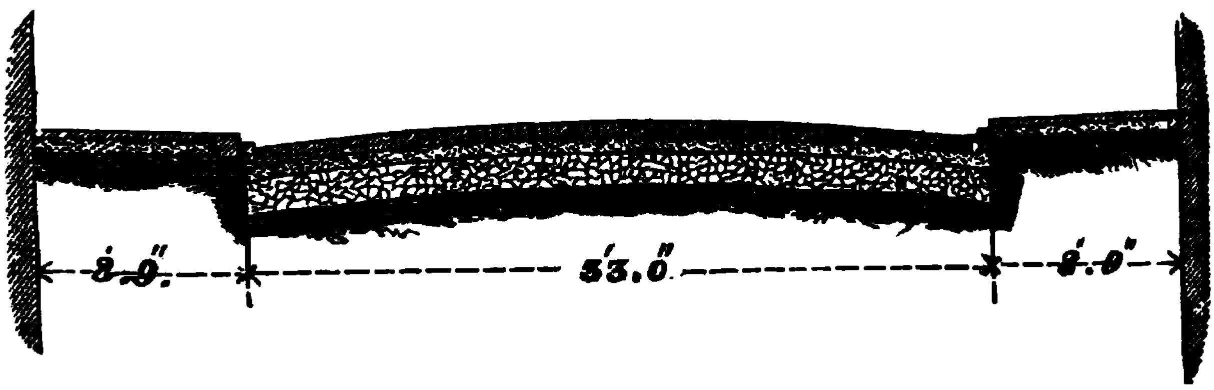


Fig. 45.—King William Street.

Granites of various qualities have been tried for street pavements in the City of London—Aberdeen, Guernsey, Herm, Devonshire, Cornish, Mount Sorrel, in Leicestershire. The harder and more durable granites—like the Guernsey and the Mount Sorrel granites—though the more economical, possess the fault of slipperiness when set in pavement. The less durable granites wear roughly, and afford a better foothold for horses. Hence it is that, for the sake of public convenience, the hardest and most durable granites are not used. Aberdeen blue granite sets have for the most part been used in the construction of City pavements; they are considered to be the best, taking together the first cost, the durability, and the absence of slipperiness.

Typical sections and plans of a 50-feet street for the City of London are shown in Figs. 46, 47, and 48. The extreme width of the street is 50 feet between the houses, divided

Fig. 48.—60-feet Street, City of London.

A

LONGITUDINAL SECTION AT C.D.

Figs. 47 and 48.—60-feet Street, City of London.

into 30 feet for the width of the carriage-way, and 10 feet for each footway. The bed of the road is excavated to a depth of 21 inches below the finished level of the street, following the contour of the surface. A layer of broken stones 9 inches thick is distributed over the ground, and is covered by a layer of hoggin or small gravel and sand

Fig. 49.—Southwark Street.

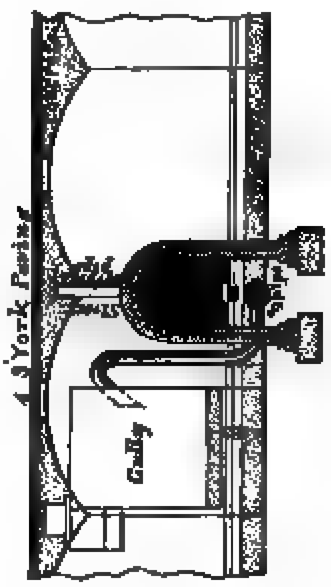


Fig. 50.—Southwark Street.

3 inches thick, as a bed for the paving. The paving consists of granite sets, or "cubes," 3 inches wide and 9 inches deep, and of length varying from 10 to 15 inches, grouted at the joints. The rise of the pavement is 6 inches for the width of 30 feet, or 1 in 30 for the average inclination; the contour being a segment of a circle. The foot-

paths are laid with 3-inch York pavement, bounded by a granite kerb 12 inches wide and 9 inches deep, showing 6 inches above the roadway pavement.

The sectional views of Southwark Street, Southwark, Figs. 49 and 50, show a good example of a first-class metropolitan street, arranged with a subway and a sewer at the middle, and cellarage at each side. The street is 70 feet wide between the houses, comprising two 12-feet footways and a carriage-way 46 feet wide. For the construction of this street the ground was levelled and the soft places cleared out. It was covered with a bottoming of brick rubbish, varying from 6 to 10 inches deep, which was rolled and bound with sand. Upon this bottom was laid a stratum of concrete 12 inches thick, consisting of blue-lias lime and clean Thames ballast, in the proportions of 1 and 6 by measure. A layer of sand or of hoggin, 1½ or 2 inches thick, was distributed over the concrete as a bed for the granite sets, which were 9 inches deep and 3 inches wide. The stones were set close and grouted together.

Colonel Haywood, in 1853, estimated the cost and the duration or life of a pavement of 3-inch Aberdeen granite sets 9 inches deep, laid in such a thoroughfare as Gracechurch Street:—

	Per square yard for 25 years. s.	d.		Per square yard for year. d.
First cost, excluding foundation .	14	6	—	6·96
Repairs: three relays at 1s. . .	3	0	—	2·04
Ditto, 20 years at ¾d. per year . .	1	3		
Total expenditure	18	9	—	9·00
Deduct value of old material . .	2	3	—	1·08
Nett total cost	16	6	—	7·92

The traffic of Gracechurch Street averaged, in 1857, above 5,000 vehicles in 12 hours.

From the best information at his command, the present writer, in 1877,* constructed the following table, showing the wear and the duration of pavement in the City of London. In this table, and the discussions which lead up to it, the attempt is made for the first time, it is believed, to measure the wear of these pavements in terms of the traffic per unit of width of pavement, a method of testing durability which has since been employed by Mr. G. F. Deacon and Mr. O. H. Howarth, in the papers read by them in 1879, at the Institution of Civil Engineers† :—

CITY OF LONDON—RECAPITULATION OF DATA ON THE WEAR AND DURATION OF ABERDEEN GRANITE PAVEMENTS.

Sets 3 inches wide, 9 inches deep.

Aberdeen granite pavements.	Vertical wear.	Duration.
	Inches.	Years.
Vertical wear for 100 Vehicles in 12 hours per foot of width per year	$\frac{1}{16}$	1
Total vertical wear in principal streets	2	15
Ditto additional ditto in minor streets .	2	20
Total vertical wear when laid aside .	4	35
Remaining depth when laid aside .	5	—
Depth of new sets	9	—

Mr. Deacon, treating of the pavements of Liverpool, states that two depths of sets are employed, 6¼ inches and 7¼ inches, for streets of moderate traffic and heaviest traffic respectively, with a width averaging 3½ inches, including interspaces. He finds that asphalt jointing is best. The sets, which should be dry, are laid as closely as possible and covered with clean, dry, hard gravel, well brushed into the joints, then rammed and recovered and re-rammed till

* *The Construction of Roads and Streets*, p. 206.

† "Street Carriage Way Pavements." By G. F. Deacon. "Wood as a Paving Material under Heavy Traffic." By O. H. Howarth.—*Proceedings of the Institution of Civil Engineers*, vol. lviii., April 29, 1879.

the pavement becomes perfectly firm. Hot asphalt, consisting of pitch and carbolic acid, or dead oil, in mixture, is run into the joints, and the surface is finished with a thin sprinkling of small sharp gravel. The cost of the pavement having sets 7½ inches deep, inclusive of foundation, amounts to 10s. 6d. per square yard, or, with 8 per cent. for general charges, 11s. 4d. For 6½ inch sets the cost is 10s. 4d. Pavements made with ordinary gravel joints, as formerly laid in Liverpool, cost respectively 10s. 7d. and 9s. 6d. per square yard, including general charges.

The foundation, as a rule, consists of a layer of Portland cement concrete 6 inches deep, composed of 40 per cent. of broken stones, 53 per cent. of river Dee gravel, and 7 per cent. of cement, costing, laid, 3s. 9d. per square yard, including general charges. Where the old bed is irregular, bituminous concrete, costing 3s. 6d. per yard 6 inches deep, is laid. It possesses an elasticity by which it is preserved from cracking, to which concrete would be liable in an uneven and occasionally thin layer.

The principal thoroughfares of Manchester are paved chiefly with syenitic granite or with trap-rock sets. The most common dimensions are 5 inches, 6 inches, and 7 inches in depth, from 3 to 3¾ inches in width, and from 5 to 7 inches long. Cubes of 4 inches were tried several years ago, but it appeared that they were unfitted for resisting the lateral stress of the traffic, particularly on streets of considerable inclination. The secondary streets are paved with millstone grit. A foundation, not exceeding 15 inches in thickness, is laid of cinder and other hard material, including three inches of gravel, as a bedding for the sets. The traffic is turned over this foundation until it becomes solid, and the temporary gravel surface is renewed from time to time. When the surface has become sufficiently solid the sets are bedded upon it and well beaten, and they are racked with clean small broken stones or with washed

gravel and filled in with an asphalt mixture of pitch made from coal tar and creosote oil. The use of this composition was originally suggested by Mr. Ronchetti, a chemist, of Manchester. The hard-core foundation, inaccessible to water, is always dry, and it has given entire satisfaction, avoiding the use of concrete.]

CHAPTER V.

WOOD PAVEMENT.

[ACCORDING to the best experience of wood paving it should consist of plain rectangular blocks solidly set upon a foundation of cement, with water-tight joints. A wood pavement so constructed as to fulfil these conditions gives satisfaction on the five points of convenience, cleansing, maintenance, safety, and durability. Unless the foundation be rigid it is impossible to maintain a pavement in sound condition; and as in macadam so in wood pavement, the dogma of elastic action has been exploded by experience, for it was found that such a degree of elasticity as is afforded by the reaction of vertical wood fibre against a vertical pressure is quite sufficient to absorb the shock of a horse's hoof and to soften the strokes of loaded wheels.

As with granite sets so with wood blocks; the gauge of a horse's hoof is the measure of the proper maximum width. The most common width of wooden blocks is 3 inches, but they are sometimes made 3½ or 4 inches wide. The normal dimensions in current practice are—width, 3 inches; depth, 6 inches; length, 9 inches. These are in the ratios of 1, 2, and 3.

The streets of the City of London afford the best and most exhaustive available experience of wood pavements. Carey's was the first durable pavement that was laid in the City—amongst other places, in Mincing Lane, in July, 1841. The blocks were from 6½ to 7½ inches wide, 13 to 15 inches

long, and 9 inches deep, on a layer of Thames ballast. They were replaced by new wood pavement in August, 1860, after having been down 19 years and 1 month. During this period the pavement was turned and relaid, and again relaid, the tops of the blocks having been cut off; and it was at other times extensively repaired. The two successive pavements lasted together 32 years. The first cost of the pavements was respectively 14s. 4d. and 9s. 2d. per square yard. Relays and repairs cost 13s. 4d. and 22s. 6¾d. per square yard. The total expenditure was at the rate of 1s. 10¼d. per square yard per year, or, deducting ¾d., the value of old material, 1s. 9½d. Of this, for maintenance alone, the charge was 1s. 1½d. per square yard per year. Carey's recent wood pavement consists of wood blocks 4 inches wide and 5 inches or 6 inches deep, according to the traffic, and 9 inches long. The ends of the blocks on Carey's system are formed with double bevelled surfaces, salient and re-entering to the extent of $\frac{5}{16}$ inch or ⅜ inch, which come together for the purpose of preventing the shifting of the blocks. The paving is laid on a bed of ballast or sand, 2 inches thick, laid on the old bed of the street, and the joints, ⅜ inch wide, are grouted with lime and sand. It is scarcely necessary to remark that this pavement can only endure when it is laid on a previously existing foundation.

The only other wood pavements that need be noticed are the asphaltic-wood pavement and Henson's wood pavement. On the former system, originally patented by Mr. Copland, a solid concrete foundation, 6 inches thick, is laid to the curvature of the road. The foundation is composed of blue lias lime and ballast in the proportion of 1 to 5 or 6. It receives a coating of mastic asphalt ⅜ inch or ½ inch thick as a bedding for the wood blocks. The blocks are 3 inches wide, 6 inches deep, and 9 inches long, of Baltic fir, laid in transverse courses, butt-jointed, with $\frac{9}{16}$ inch interspaces

run up with melted asphalt to a depth of 1½ inches, and filled with a grouting of sand and hydraulic lime. The surface is finished with a sprinkling of small gravel. This pavement has answered satisfactorily. After having been two years down the foundation was found to be water-tight.

Henson's wood pavement is laid on a solid substratum of blue-lias lime concrete, 6 inches thick, covered by a 2-inch layer of cement concrete of a finer quality, upon which a coating of ordinary roofing felt is spread, the felt having previously been saturated with a hot asphaltic composition of distilled tar and mineral pitch. On this felt, as on a carpet, cushiony and impervious to moisture, blocks of Swedish yellow deal of the ordinary dimensions, containing resin sufficient for preservation, are laid closely together, end to end, in rows across the street. The rows are also driven together and close-jointed with a strip of saturated felt in each joint. The width of the interspaces is thus limited to the simple thickness of the felting, and does not exceed if it even amounts to ¼ inch. At intervals of every three or four rows, a row of blocks grooved across the middle is laid to aid in giving foothold. The surface is dressed with a hot bituminous compound and fine clean grit.

Mr. Howarth gives the cost of the Henson pavement as 11s. 6d. per square yard: comprising red deal blocks, 5s. 6d. per square yard; felt, 6d.; Portland cement concrete, 2s. 6d.; labour, watching, lighting, and all extras and dressing, 3s.; in all, 11s. 6d. per square yard. He estimates the cost for maintenance in perpetuity at 2s. 5d. per square yard per year.

The wear of wood paving has been estimated by the writer, from the results of observation, to average 0·30 inch per year vertically for an average traffic of 362 vehicles in 12 hours per foot of width: equivalent to $\frac{1}{12}$ inch per 100 vehicles per day per foot of width; or to one-third more than granite sets as before estimated.]

CHAPTER VI.

ASPHALT PAVEMENTS.

[ASPHALT pavements were first laid in Paris, where, in 1854, the Rue Bergère was laid with Val de Travers asphalt. In 1858, three sides of the Palais Royal were laid with the material, which was brought to the ground in the state of rock crushed into small pieces, and was heated and powdered by decrepitation. On a foundation of concrete, 6 inches thick, from 2 to 2·4 inches of asphalt was laid, at a cost of 13s. 4d. per square yard. The conversion of street pavements into asphalt work on a large scale was commenced in 1867.

In the City of London, carriage-ways are constructed with Val de Travers compressed asphalt, on concrete foundations of from 6 to 9 inches in thickness, according to the traffic. The rock in its natural state is broken up and reduced to powder by exposure to heat in revolving ovens. It is then lodged in iron carts with close-fitting covers and brought to the ground, taken out, laid over the surface, and whilst hot compressed with heated irons into a homogeneous mass without joints. The finished thickness is from 2 to 2¼ inches, according to the traffic; and the material is further compressed and consolidated by the action of traffic by as much as 20 or 25 per cent., according to the statements of the company. The first asphalt pavement was laid in Threadneedle Street, near Finch Lane, in May, 1869. The next pavement that was laid—in Cheapside and the Poultry—was

2¼ inches thick, on 9 inches of concrete, costing 16s. 3d. and 1s. 9d. respectively per square yard; together, 18s.

Other varieties of asphalt pavement have been laid in the City of London, all of them inferior to the pavement just described.

The wear of Val de Travers asphalt pavement has been estimated by the writer at ⅓ inch for 208 vehicles per day per foot of width.]

CHAPTER VII.

RAILWAYS.

[Railways, like common roads, should be laid out with regard to the circumstances of the locality to be provided with them, and the selection of the route is governed for the most part by the same leading principles. At the same time there are different influences in operation. The considerable cost of the rails renders it of greater importance to shorten the length of the route than to make slight savings in earthwork. As an artificial bearing surface of rails and sleepers is provided, the state of the natural surface of the ground passed over is of less consequence for a railway than for a common road. A common road should be laid out as nearly on the surface as is practicable for the purpose of giving access to the adjoining lands, whilst, for railways, means of communication with the neighbourhood is only required at intervals selected with reference to local circumstances, and it is for the most part immaterial whether between the stations the line lies on the natural surface or otherwise.

Sharp curves and steep gradients are evils, involving special extra cost for maintenance and for working, although the original outlay may be economized by the adoption of them.

Gauge of Railways.

The measure of the standard gauge of railways is 4 feet 8½ inches width between the rails. There are many other gauges in existence in different parts of the world. In Eng-

land, the 7-feet gauge was originally adopted on the Great Western system, and was known as the "broad gauge" in contradistinction to the ordinary gauge of 4 feet 8½ inches, which was for a long time known as the "narrow gauge." Nowadays the 7-feet broad gauge is replaced by the standard gauge; and, in consequence of the altered conditions, the name of narrow gauge has ceased to be applicable to the standard gauge, and is applied only to gauges of much less width—the metre gauge and others of from 2 feet to 3 feet or 3½ feet wide. The following statement comprises the gauges of the principal railways in all parts of the world, with a few exceptional instances of local interest:—

	Ft.	In.
Great Britain, Standard Gauge	4	8½
,, Festiniog Railway	1	11½
,, Talyllyn Railway	2	6
Ireland, Standard Gauge	5	3
Europe, Prevailing Gauge	4	8½
Russia, Standard Gauge	5	0
Norway	4	8½
	3	6
Spain, Standard Gauge	5	6
Antwerp and Ghent	2	3
India, Prevailing Gauge	5	6
Do. State Railway Gauge (1 metre)	3	3⅜
Do. Arconum and Conjeveram Railway	3	6
Japan	3	6
United States, Prevailing Gauges	4	8½
	4	9
United States	6	0
	5	6
	5	0
	3	0
Canada	5	6
	4	8½
	3	6
Chili	5	6
	4	8½
	4	2
	3	6
Brazil	5	3
South Australia	5	3
	3	6
Queensland	3	6
New South Wales	4	8½
Victoria	5	3

The relative advantages of broad gauges and narrow gauges were exhaustively discussed at the Institution of Civil Engineers, on the reading of Mr. W. T. Thornton's paper on "The Relative Advantages of the 5 feet 6 inch Gauge and of the Metre Gauge for the State Railways of India." * Nearly all that could have been urged in favour of very narrow gauges was well considered and refuted in the course of the discussion. The fallacy pervading the arguments for narrow gauges is, that the gauge or width apart of the rails on which the wheels are to run is the basic unit of the system, whereas the width of gauge is little more than an incident. The basic unit is the capacity of the vehicles required for the carriage of the traffic. Hence the magnitude and weight of the vehicles govern the dimensions of the railway, not the incident of gauge. It appears that, taken altogether, the "normal" or "standard" gauge, of 4 feet 8½ inches, is certainly not less efficient than, and is at least as good as, any other gauge, for the purposes of general traffic.

The several gauges above noted are here represented graphically, in their proportions. The old 7-feet gauge of the Great Western Railway is added for comparison :—

	Ft.	In.
	1	11½
	2	3
	2	6
	3	0
(Metre) . . .	3	3⅜
	3	6
	4	2
Standard . . .	4	8½
	4	9
	5	0
	5	3
	5	6
	6	0
	7	0

* *Minutes of the Proceedings of the Institution of Civil Engineers*, 1872-73, vol. xxxv. p. 214.

Cuttings and Embankments.

The width of embankments is regulated rather by the weight and the speed of the trains than by the width between, or gauge of, the rails. The width of cuttings must be regulated by the width of the rolling-stock, and by the space required for drainage outside the permanent way. Engineers endeavour so to plan the works of a railway that the earth to be excavated shall be equal in volume to the embankment, effecting a redistribution of material rather than its removal, and arriving at the desired result by the simplest means and in the most economical manner.

The earthwork is the foundation and support of the whole superstructure, and, as such, must be uniformly firm and carefully considered with respect to material, preparation, form, and drainage. Fig. 51 shows in section the ordinary formation

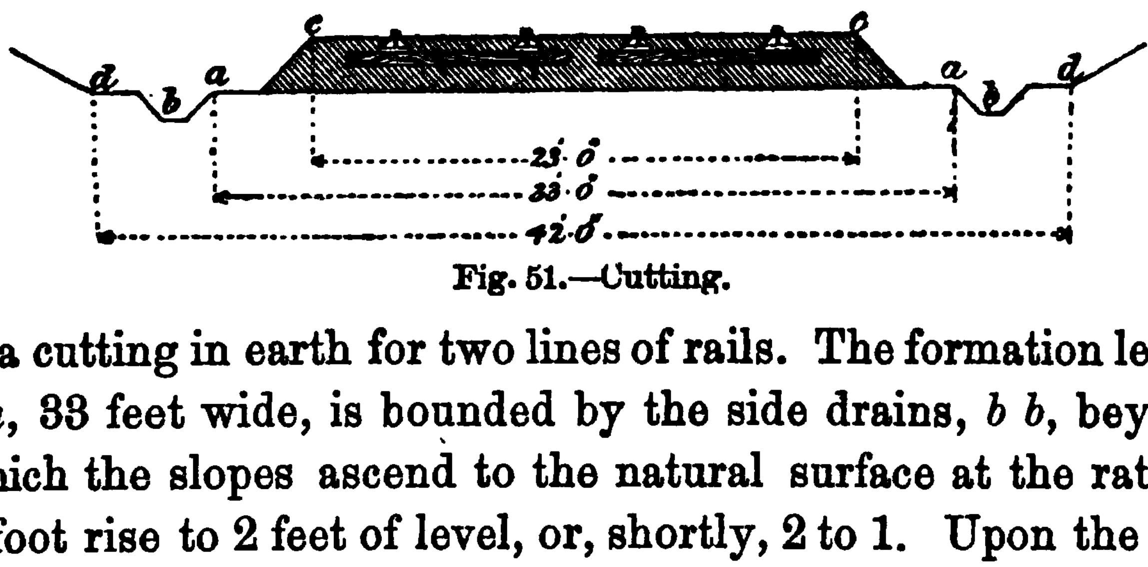


Fig. 51.—Cutting.

of a cutting in earth for two lines of rails. The formation level, *a a*, 33 feet wide, is bounded by the side drains, *b b*, beyond which the slopes ascend to the natural surface at the rate of 1 foot rise to 2 feet of level, or, shortly, 2 to 1. Upon the formation level, the ballast, *c c*, is deposited, 2 feet in depth, and about 23 feet wide at the top—being so wide, in fact, as to extend 4 feet on each side beyond the outer rails. The sleepers and the chairs are buried in the ballast, and the rails partially also, the latter standing 2 or 3 inches above the ballast. The total width of cutting at the base, *d d*, is 42 feet. At the top, of course, the width varies with the depth of the cutting. Embankments, Fig. 52, are usually the same as cuttings in their ruling dimensions, the forma-

tion level being, as in the other, 33 feet wide, sloping down to the natural surface. The dimensions are suited for the

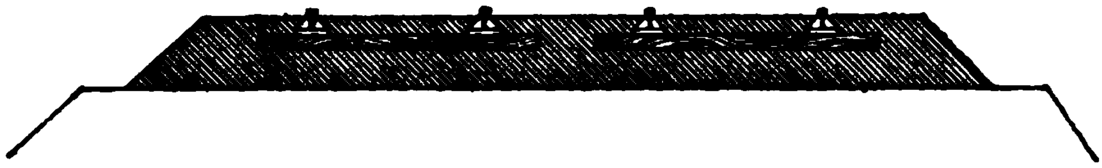


Fig. 52.—Embankment.

standard gauge, 4 feet 8½ inches. A clear space, 6 feet wide, is allowed between two lines of rails.]

The cuttings on the London and Birmingham Railway have afforded much useful information. One of the cuttings on that line, near Cow Roost, through a very wet white chalk, although only 25 feet in depth, required a slope of 1½ to 1, while that at the north end of the Watford Tunnel, although consisting of soft wet chalk mixed with flints, stands with a slope of ¼ to 1; as does also the cutting, 35 feet in depth, through chalk, chalk-marl, and gravel, at the north end of the Tring Tunnel. One of the most interesting, however, of the cuttings on this line is that near Blisworth, a section of which is shown in Fig. 53. In this case, a stratum of limestone rock, about 25 feet in thickness, was found about the centre of the cutting (vertically), having looser strata both above and below it, and the difficulty to be overcome was to prevent the latter, consisting of wet clay, from being forced out from the weight of the superincumbent mass of rock, which was very successfully done in the following manner: a rubble wall, on an average 20 feet in height, was built on each side, underneath the rock, in the manner shown in the figure, strengthened by buttresses at every 20 feet, resting on inverts carried under the line. Behind these walls a puddle drain was formed with a smaller drain through the wall, by means of which the water was led off from the wet strata immediately beneath the rock. The right-hand half of the section is taken through the wall

between two of the buttresses, and the left-hand half through one of the buttresses and the invert; the method here adopted is technically called *undersetting*. The rock itself is cut to a slope of ½ to 1, and the strata above it to a slope of 2 to 1, a bench 9 feet in width being left on the upper surface of the rock.

The Newcastle and Carlisle Railway affords an example of a cutting 110 feet in depth, through clay intermixed with veins of sand, standing with a slope of 1½ to 1. This

Fig. 58.—Cutting, at Blisworth.

cutting is through the Cowran Hill, and the lower part, to the height of 14 feet, is supported by a stone retaining wall, having an open drain along its summit, which receives the water from the surface of its slope.

A remarkable instance of the tendency of some kinds of ground to slip has been afforded by the cutting (nearly 100 feet in depth) through the London clay on the London and Croydon Railway, near New Cross. The slopes were finished at 2 to 1, and stood (with the exception of a few small

slips) at that inclination for about two years, when, after a succession of wet weather, they suddenly commenced slipping to such an extent that the line was rendered impassable for some weeks, and some parts of the slopes had to be reduced to an inclination of 4 to 1.

Many different methods have been suggested and adopted to prevent slips from taking place. But one of the simplest means is thorough drainage, without which the best description of ground will in time be acted upon by the combined action of land springs and the weather.

With regard to embankments, although less uncertainty exists as to the slopes at which different descriptions of ground will stand, still this depends to a very great degree upon the nature of the ground supporting the base of the embankment, as well as the state of the weather, and the care and attention bestowed upon it during its formation.

Many embankments have failed in consequence of the ground upon which they have been formed not being sufficiently firm and solid to support the large additional weight thus brought upon it; to prevent this cause of failure, it is desirable to form very high embankments of the lightest material that can be obtained, to extend the base of the embankment, and, if the ground upon which it is to be formed is soft and saturated with water, thoroughly to drain it previous to forming the embankment. A remarkable instance of the failure of an embankment from this cause was afforded in the case of the Newton Green embankment, on the Sheffield and Manchester Railway, which subsided to such an extent that the base of the embankment spread out to two or three times its original width, and it was found necessary at last to carry the rails across those parts which had slipped, upon timber shores.

A striking instance of the success of the means which we have enumerated for carrying embankments over loose ground has been afforded by the construction of the Liver-

pool and Manchester Railway across Chat Moss, by the late Mr. George Stephenson. In this case the ground was of so soft a nature that cattle could not walk upon it, and an iron bar sunk through it by its own weight, the moss being in many parts not less than 34 feet in depth. That portion of the moss upon which the embankment (in some parts as much as 12 feet in height) was formed was first thoroughly drained by deep drains cut parallel to the intended line of the railway; and, when this had been properly effected, the embankments were formed of the lightest material which could possibly have been employed, namely, of the dried moss itself. Had the usual heavy materials, such as clay and gravel, been employed, their weight would have caused them to sink through the moss until they reached the firm ground beneath, and the quantity which would have been required would have been immense; as was found to be the case upon the same line, where, an embankment only 4 feet in height having been formed over a smaller moss of a similar description, the quantity of clay and gravel employed would have formed an embankment 24 feet in height on firm ground.

The slopes of both cuttings and embankments, as soon as they have been trimmed to their proper form, should be covered with soil, and sown with rye-grass and clover seeds mixed, which soon spring up, and form a very effectual protection from the influence of the weather.

CHAPTER VIII.

PERMANENT WAY OF RAILWAYS.

We come now to describe the manner in which the *Permanent Way* (as it is technically called) is formed, that is, the rails by which the carriages are guided and prevented from deviating from the line of the railway; and in doing so we must not omit to notice the *tramplates* which were at first adopted, and which have now universally been superseded by the *edge rail.*

The essential difference between a railway and a common road consists in forming a smooth narrow surface for the wheels of the vehicles to run upon, with the means of preventing them from deviating from the track thus formed. Two different modes of effecting this have been adopted, which are shown in Figs. 54 and 55. By the first method (Fig. 54), the path for the wheels is formed by iron plates, and they are prevented from running off these plates by a flange *a*, formed on their outer edge; these are termed *tramplates*, and a road so formed is called a *tramway*. This method has, however, been generally superseded by that shown in Fig. 55, where the track for the wheels is formed by a narrow bar of iron, placed edgewise, in consequence of which it is termed the *edge rail*, and the road

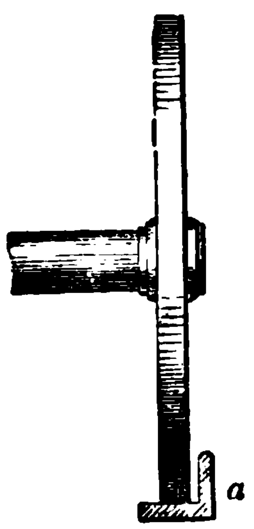


Fig. 54.

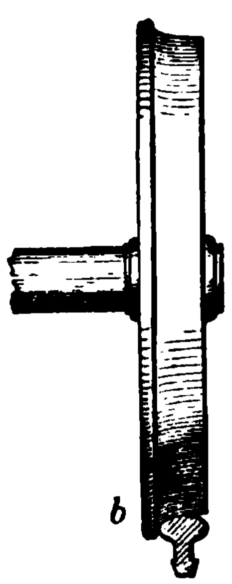


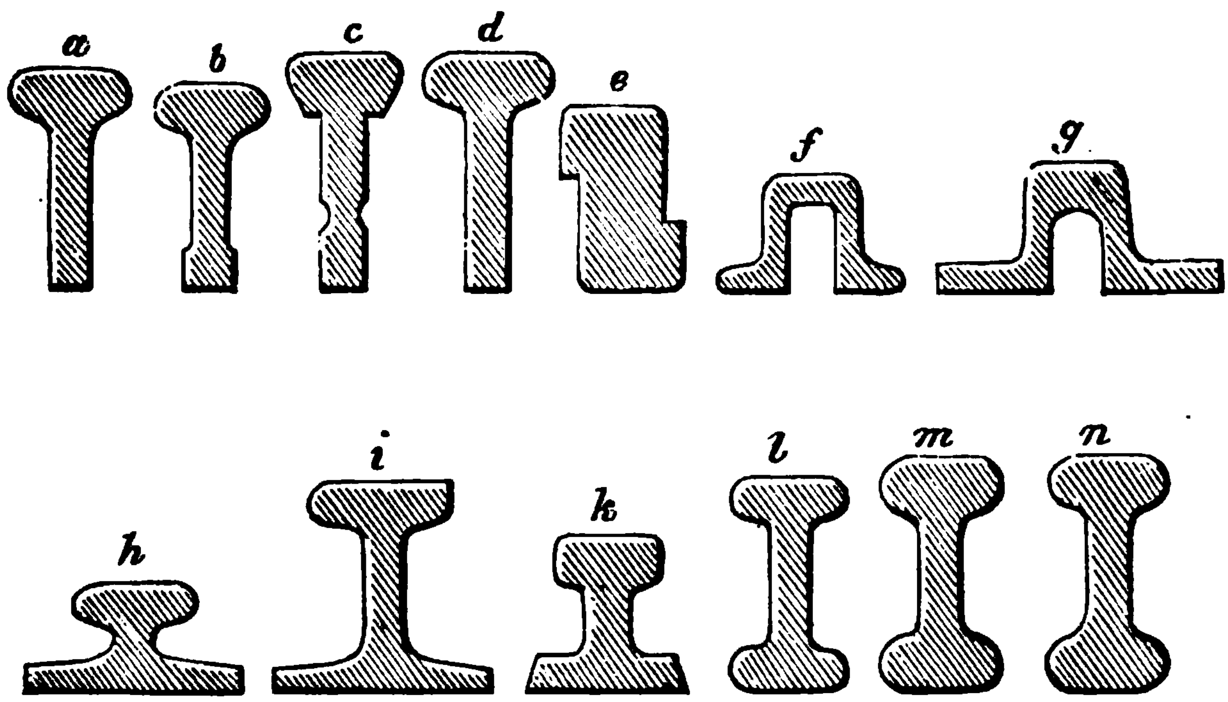
Fig. 55.

formed with them a *railway;* in this case the flange *b* for guiding the wheel is placed upon the wheel itself instead of on the rail. In comparing the two methods, it will soon be seen that the railway possesses many advantages over the tramway. In the latter, the wheels are only prevented from running off the tramplates by coming in contact with one or other of the flanges on their edges, while in the former a very simple and beautiful means (which we shall describe presently) has been devised by which the wheels are preserved in their proper position on the rails without their flanges coming in contact with the rails at all—a circumstance which only occurs when any unusual force solicits the carriage to deviate from its proper course. The effect of the wheels thus coming into contact with the edges of the trams is to cause a great additional resistance to the motion of the carriages, and consequently a large additional cost in overcoming it. Another disadvantage is, that the angle of the tramplate formed by the raised flange is very likely to become filled with rubbish, by which the friction of the wheels is still further increased.

A great many different forms of rails have been adopted, a few of which are shown in Figs. 56; the names of the railways on which they have been employed, their weight in pounds for every yard in length, and the distances apart at which they are supported, being shown in the following table:—

Reference to Figs. 56.	Name of railway.	Distance of chairs apart.	Weight in lbs. of 1 yard in length of the rail.
		Ft. Ins.	
a	Durham and Sunderland . .	3 0	42
b	Berlin and Potsdam . . .	..	52
c	London and Blackwall . .	..	56
d	Manchester and Birmingham .	..	65
e	Saint-Etienne to Lyon (New) .	3 6	50
f	Wilmington and Susquehanna .	..	40
g	Great Western	Continuous Bearing.	44 to 62
h	London and Croydon	Id.	55
i	Morris and Prevost	..	56
k	Birmingham and Gloucester .	2 6	56
l	London and Birmingham . .	3 9 to 4 0	65 to 75
m	London and Brighton	3 9	76
n	Midland Counties	5 0	77

It should be stated that the upper surface of each of the rails shown in the Figures 56 is made slightly rounding, the



Figs. 56.—Old Sections of Rails.

object of which we have now to explain. On a common road or on a tramway the wheels are *cylindrical*, that is, the diameter of the wheel is the same both on its inner and

outer sides, as shown in Fig. 57; but upon a railway the wheels are made slightly *conical*, as shown in Fig. 58, so that the diameter (A B or C D) of the wheel on its outer side is about half an inch less than its diameter (E F or G H) on the inner side near the flange. Now the effect of this difference in the inner and outer diameters of the wheel is to keep the wheel in its proper position in the centre of the railway, and to prevent the flanges of the wheels from coming into contact with the rails unless under extraordinary circumstances, such as a very strong side wind or a sharp curve. In Fig. 59, the wheels of the carriage are represented as being thrown over on one side, so that the flange of the right-hand wheel has been brought nearly to touch the rail. Now if the

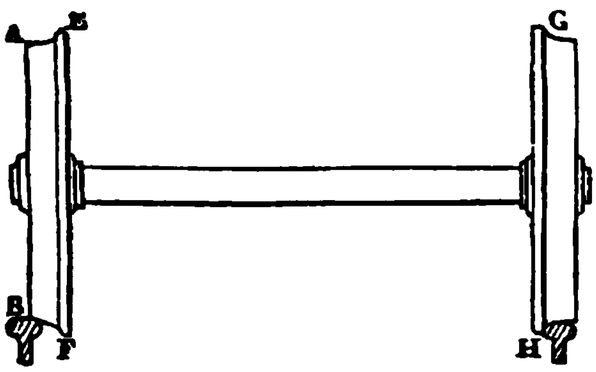


Fig. 57.

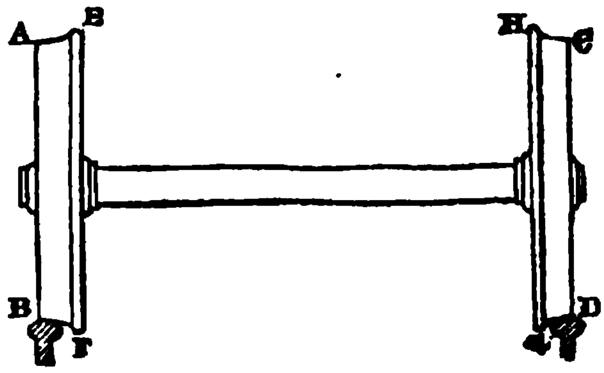


Fig. 58.

wheels were cylindrical, and the force which had caused the carriage to swerve in the manner shown in the figure were still to continue in action, the flange would be brought into actual contact with the rail, and would so remain until the force ceased, or until some greater force solicited the carriage to swerve in the opposite direction; but if we carefully examine the diagram, we shall perceive that the deviation of the carriage to the right has brought the outer and smaller diameter of the wheel A B to bear upon the left-hand rail, while the inner and larger diameter of the wheel G H is brought to bear upon the right-hand rail, for in consequence of the upper surface of the rail being slightly rounding, the wheel only rests upon it in one point. With a displacement equal to that shown in the figure, the difference of the dia-

meters of the wheels would be about three-quarters of an inch, which would cause a difference in their circumferences of upwards of two inches; and as the distance that each wheel would advance upon the rail in one revolution would be equal to its circumference, and the two wheels being firmly fixed on to the same axle are obliged to revolve together, it follows that, for every revolution that they make, the right-hand wheel will advance two inches more than the left-hand, and quickly restore the carriage to the position shown in Fig. 59, where the diameters of the wheels being the same, the carriage has no tendency to move towards either side.

This self-adjusting action of the conical wheels is found sufficient to preserve the carriages in their proper position upon the rails on those portions of the line which are rectilinear or straight; but on the curved portions a new force, the centrifugal force, is called into play, by which the carriage is solicited to move in a straight line; and if the radius of the curve is less than a certain limit, the mere action of the conical wheels is not sufficient to counteract this tendency of the carriages to move in a straight direction, and to cause them to follow the course of the required curve. To effect this, therefore, and prevent as much as possible contact between the flanges of the wheels and the rail, another means has been devised of throwing the carriages over to the opposite side to that on which the centrifugal force tends to keep them. This means consists in raising the rail on the outer side of the curve to a certain height above that on the inner side, by which the carriage is thrown over in the position shown in Fig. 59, and a tendency given to it to slide

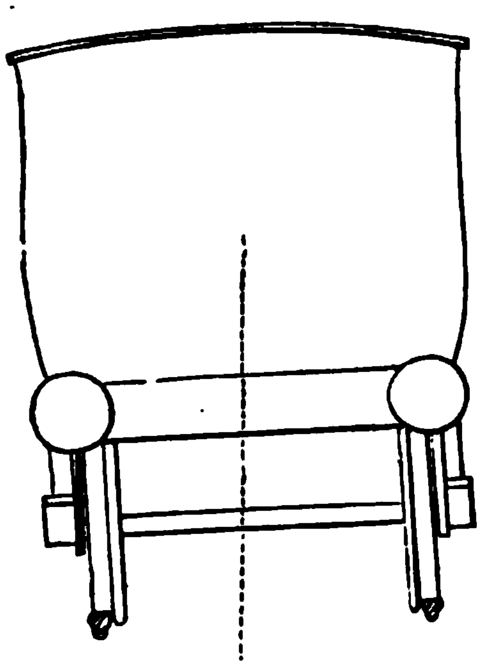


Fig. 59.

towards the inner side, the height, or, as it is termed, the *superelevation*, of the outer rail being so adjusted that this tendency, combined with the effect of the conical wheels, is just sufficient to balance the centrifgual force.

If V = the velocity of the train in feet per second,

r = the radius of the curve in feet,

Then the centrifugal force of the whole train is to its weight in the ratio of

$$\frac{V}{32{\cdot}2\,r} : 1$$

and this is the ratio which the cant or vertical height of the outside rail above the level of the inside one should bear to the width of gauge.

Hence, if V = the speed of the train at maximum in miles per hour and B = the breadth of gauge ; K = the cant.

$$K = B + \frac{V}{15r}$$

At 40 miles per hour, the cant is for the British and Irish gauges as follows :—

4 ft. 8½ in. . . . $\frac{6{\cdot}00}{r}$ inches

5 ft. 3 in. . . . $\frac{6{\cdot}72}{r}$

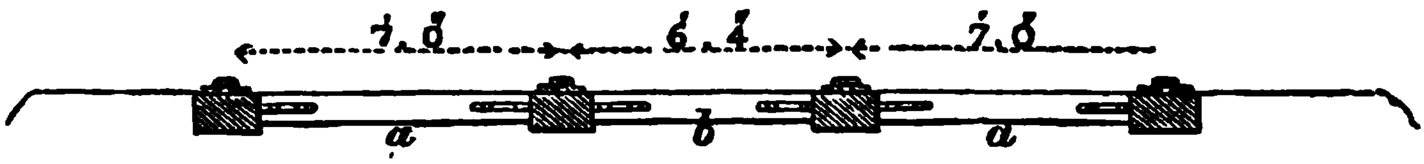
It will be seen that the steadiness of the carriages composing a train must be very considerably affected by any variation in the distance between the rails, or in the height of one rail above the other when not intended to counteract the effect of a curve ; and the importance of laying the rails and sleepers (that is, the permanent way) in the most solid and substantial manner will be at once perceived. With the view of attaining this end, several different methods have been devised for fixing and supporting the rails ; these may all, however, be generally classed under two heads, viz. those having a continuous bearing, or in which the rails rest upon wooden sleepers throughout their entire length, and

those which are only supported at certain intervals (varying from 2 feet 6 inches to 5 feet as given in the table at page 49) on metal chairs, as they are technically termed.

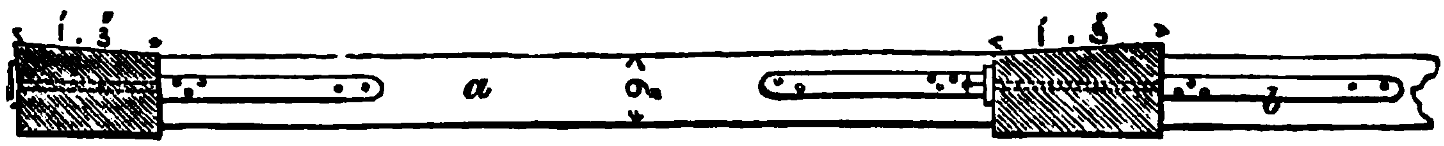
The Great Western was the first line on which the continuous bearing was employed, this method of laying the rails having been suggested by Mr. Brunel, who was the engineer of that line. The method there adopted is shown in Figs. 60; the rails (the form of which has been already given at *g*, Figs. 56) are firmly screwed to a piece of timber 15 inches in width, 7½ inches in depth on the outer side, and 7 inches in depth on the inner, by which means the rail is made to slope somewhat inwards to counteract the spreading tendency produced by the conical wheels. A piece of patent felt is interposed the whole way between the rail and the timber, forming an elastic bed for the rail. The longitudinal timbers are preserved at their correct distance apart by transverse pieces (*a a*) placed between them at every 11 feet, being notched into the timbers on both sides, and further secured to them by wrought-iron knee-traps. Similar pieces (*b b*) are also placed at distances of about 14 feet apart, between the two lines of railway, in order not only to preserve them at their proper distance, but to steady the whole. The ground immediately under the sleepers, and upon which they bed (technically called the *ballasting*), should be composed of clean gravel, broken stone, burnt clay, or any other hard material not affected by wet; it should be well rammed and packed under the rails, and its upper surface should be formed in the manner shown in the section, so as to lead off any water which may fall upon it, and prevent its soaking through to the timber. The continuous bearing was adopted on other lines besides the Great Western.

The system, however, which has been most generally adopted is that of fixing the rails in iron chairs supported

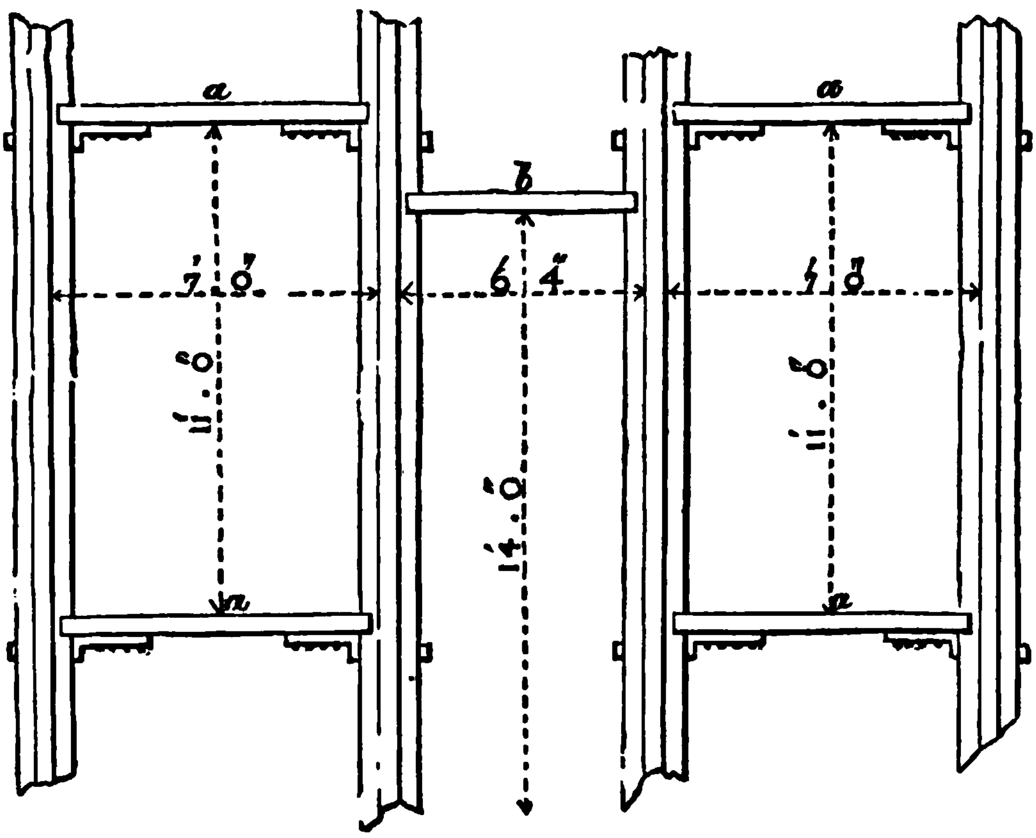
Section.



Permanent Way in Embankment.



Plan.



Figs. 60.—Permanent Way, Great Western Railway.

upon sleepers placed at certain intervals. The system shown at A, Fig. 61, is the mode in which the London and Birmingham and many other lines were laid in those portions which were in cutting, and it consists in fixing the chairs supporting the rails to blocks of stone, usually from 4 to 5 cubic feet in bulk, which are firmly embedded in the ground; they are most frequently laid diagonally, as shown at A. This method has, however, been in a great measure superseded, and timber sleepers are almost universally employed at the present time. The form of timber sleeper most universally employed is that shown at B, being a piece of round timber between nine and 10 feet in length, and about 12 inches in diameter, sawn down the middle and laid with the flat side downwards, a flat bed being adzed out on the upper side for each of the chairs. Another form of sleeper (as shown at C) was employed by Sir William Cubitt on the South Eastern Railway, which consists of a piece of square Baltic timber sawn twice diagonally, as shown at D, so as to produce

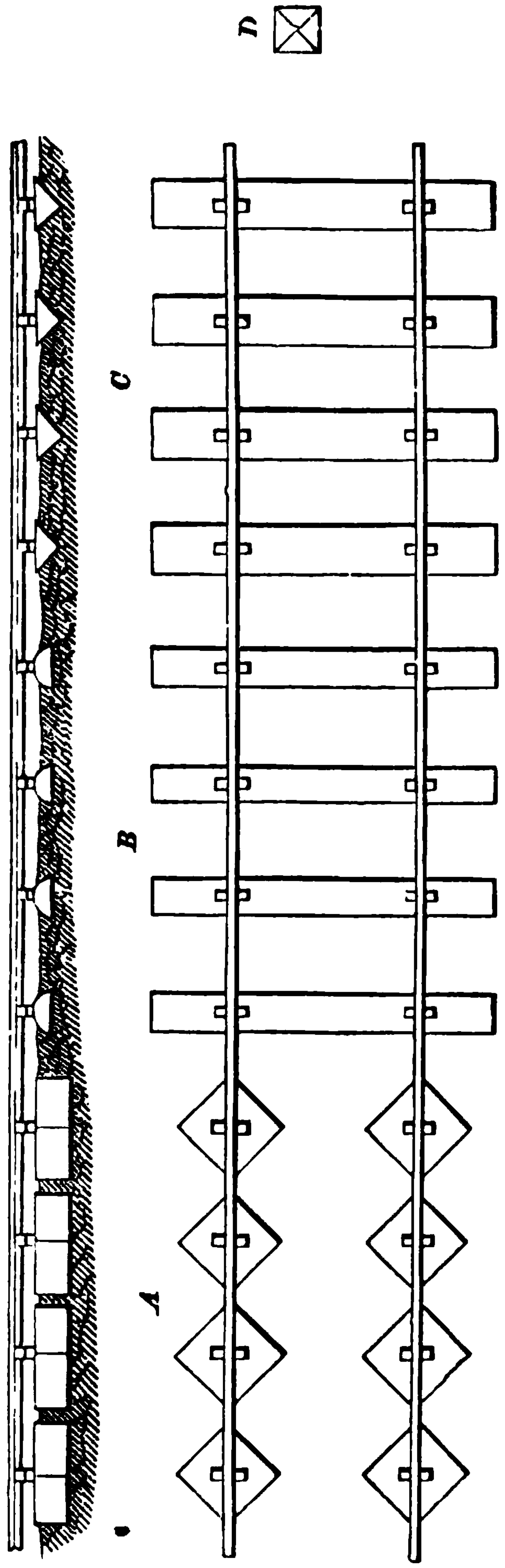


Fig. 61.—Old Systems of Permanent Way.

four sleepers, which are laid with their broad flat face uppermost.

Rails.

[The rails now generally, indeed, universally used for the way of railways are the double-headed rail, Figs. 64 and 65, and the flanged or Vignoles rail, Figs. 68 and 69; the former being keyed into cast-iron chairs spiked to sleepers, the latter being laid upon and fastened direct to the sleepers. The principal advantage of the flange rail is the facility with which it can be attached to the sleeper with fastenings of a simple description. The disadvantages are that it cannot be turned when the head is worn, as the double-headed rail may be, that the flanges are apt to oxidize and laminate, and that the rigid attachment of the rail to the sleeper causes a greater degree of disturbance of the way and involves more labour for maintenance than in the case of the double-headed rail.

The double-headed rail is somewhat heavier for the same weight of train than the flange rail. It is easily bent into curves, whilst the mode of attachment to the chairs, by wooden keys, admits of a longitudinal movement of the rails. With such freedom, the sleepers are liable to be dragged backwards and forwards in the ballast.

With regard to the dimensions of the double-head rail, the width of the table should not be less than 2 inches, nor need it exceed $2\frac{1}{2}$ inches, whilst the depth of the rail, from the nature of the fastenings, cannot well be made less than 5 inches, the depth usually adopted, and it need never exceed $5\frac{1}{4}$ inches. The minimum weight of the ordinary double-headed rail is about 70 lbs. per lineal yard; the maximum weight need never exceed 84 lbs. per yard. The chairs should be made broad in the seat, and about 25 lbs. in weight, and placed on sleepers 3 feet apart. The keys are placed on the outside of the rail, as by this arrangement

the jar is lessened between the rail and the chair. When the rails are keyed on the inner side, the keys are exposed to be crushed by the wheel-flanges when the tread of the tyres becomes worn. At the same time, in tropical climates, it is advisable to place the keys on the inner sides of the rails; for, by reason of the wide range of temperatures to which the keys are exposed in contact with iron, they are subject to expansion and contraction, in so much that they are likely to be shaken out of the chairs if preventive means be not employed. The general question, as Mr. C. B. Lane puts it, involves too many independent variables to be embraced by any single formula.

The minimum weight of the ordinary flange-rail is about 45 lbs. per lineal yard. If the weight is made less than this for main lines, the bearing surface of the rail is objectionably narrow, and comes too close to the surface of the sleeper.

"The experience of the last twenty-five years," said Mr. Bidder, speaking in 1861, "has shown that one system has been adopted almost universally—the double-headed rails, upon chairs, with cross sleepers—a plan which has been materially improved by fishing the joints. The ingenuity of inventors has been exercised, and the bridge rail and many other descriptions have been introduced; but none of these has met with universal success. His own conviction was, that the double-headed rail, when of proper materials, with properly proportioned chairs, and properly fished, was the safest and the nearest approach to perfection that could be practically attained in climates like that of England," *

It is curious to note the difference of opinion amongst engineers as to the best form of rail section. On the continent of Europe and in America, engineers have almost universally laid the flat-foot rail; and in France, double-headed rails, keyed in chairs, have been replaced by flat-foot rails. On the Metropolitan and Metropolitan District Railways,

* *Proceedings of the Institution of Civil Engineers*, vol. xx. p. 290.

on the contrary, the flat-foot rails have been taken up and replaced by double-headed, or rather bull-headed, rails, in chairs.

The case may be briefly stated in the following terms. The double-headed rail system, with chairs, is the best system, where supplies of materials and labour for maintenance and repair are always ready and available. The single-headed flange-rail is the best system where the greatest degree of simplicity and economy in construction and in maintenance constitute the chief consideration.

The substitution of steel for iron as the material for rails has become very generally practised. As Mr. R. Price Williams, a leading authority on permanent way, writing in 1876, justly remarks, "Having regard to the fact that, ten years ago, the life of iron rails, on some of the most heavily worked lines of railway, was barely three years, it is questionable whether now, with three times the amount of traffic, it would be possible to carry it on without steel rails."* He deduced, from the averaged results of observation and experiment, that the vertical wear of steel rails was at the rate of $\frac{1}{16}$ inch for 30,000,000 tons passed over them; and it appears from his deduction that a fully-proportioned bull-head rail of steel would last out 15 or 18 iron rails. The greater durability of steel rails is not due merely to their greater strength or hardness, but very much to the fact that the material of a steel rail is homogeneous and fibreless, and that it holds together so long as there is any of its substance left, as it is worn away by simple wear. Iron rails, on the contrary, fail by disintegration,—a separation of the strands or faggots of which they are composed,—cemented together, not welded, after the superficial coating is worn off.

Mr. Langley, in the course of the discussion on Mr. Price

* See Mr. R. Price Williams's paper "On the Permanent Way of Railways."—*Proceedings of the Institution of Civil Engineers*, 1876, vol. xcvi. p. 147.

Williams's paper, mentioned the results of his experience on the Blackwall branch of the Great Eastern Railway. In 1874 he laid down some permanent way near Stepney Station, where there were upwards of three hundred trains a day passing over a single line. The weight of each train was, on an average, about 150 tons, making a total of about 45,000 tons daily over one line of rails. The permanent way on this length was composed of both steel rails and iron rails (supplied by nearly all the principal manufacturers in England), weighing 80 lbs. to the yard, and keyed in cast-iron chairs resting on rectangular sleepers. The steel and the iron rails were purposely laid close together, so as to be under precisely like conditions of wear and tear. The greater number of wrought-iron rails had to be turned in one year and three quarters, during which period they had worn down about $\frac{1}{8}$ inch; but the necessity for turning them did not arise from the wear itself, but because they gave way in places, either by bulging or by splitting. The steel rails had worn about $\frac{1}{16}$ inch in the same period of one and three quarter years; about 27,000,000 of tons had passed over them—rather less than the tonnage given by Mr. Price Williams. "The fact of the wrought-iron rails," Mr. Langley justly observes, "wearing away twice as much as the steel, was not an indication of the true value of the two rails; because the steel rails, after wearing down $\frac{1}{16}$ inch, were still available, and would continue to be so until nearly the whole of the head was worn off, the wearing-down being regular and uniform. The iron rails, on the other hand, were crushed in places, and no longer fit to remain in the road." Mr. Langley mentioned the results of other comparative trials he had made at the Nine Elms goods-yard, on the London and South-Western Railway. In February, 1873, he had laid a steel rail on one side, and an iron rail on the other side, of the shunting-road where there was most traffic—an average of nearly four hundred engines and trains passing over this line

in the day of twenty-four hours. The steel rail was, at the time of speaking, in May, 1876, still in good condition—a layer $\frac{3}{16}$ inch thick having been worn off; whilst, during the same period—over three years—the iron rail on the opposite side had been renewed three times, the renewal having taken place after each rail had been turned, and both heads so worn that the rail was unfit for further service.

The cost of relaying one mile, single line, of the principal English railways, during the period 1865—75, is given by Mr. R. Price Williams as follows :—

Double-headed rails, average weight 80 lbs. per yard; price £7 10s. per ton.

	Yards.	Lbs.	Tons.	Cwts.	£	s.	d.	£	s.	d.	£	s.	d.
Rails,	3,520 at	80 =	126	0 at	7	10	0 =	945	0	0			
Chairs,	4,024 „	36 =	65	0 „	3	15	0 =	243	15	0			
	Pairs.												
Fishplates,	503 „	24 =	5	10 „	7	10	0 =	41	5	0			
Bolts,	2,012 „	1½ =	1	7 „	10	15	0 =	14	10	3			
Keys, 4,024 .				„	3	15	0 =	15	1	10			
Trenails, 8,048 .				„	3	15	0 =	30	3	8			
Spikes, 4,024 .			1	16 „	9	0	0 =	16	4	0			
Sleepers, 2,012 .				„	3	0	0 =	301	16	0			
Labour, 1,760 yards .				„	0	0	11 =	80	13	4			
								1,688	9	1			
Renewal of top ballast, 1,792 yds. at 3s. 6d.							=	224	0	0			
Cost per mile, single line . . .								1,912	9	1			
Old rails . . .			105	6 at	3	15	0 =	394	17	6			
Old chairs . .			37	0 „	2	5	0 =	83	5	0			
Fishplates . .			4	19 „	6	17	6 =	34	0	8			
Wrought scrap iron .			1	0 „	4	12	6 =	4	12	6			
											516	15	8
Nett cost per mile, single line											1,395	13	5

CHAPTER IX.

STANDARDS OF PERMANENT WAY OF RAILWAYS.

[The standard model of permanent way, on the double-headed rail and chair system, adopted by Mr. John Fowler, is illustrated in Figs. 62 and 63, showing the formation, the ballast, the sleepers, and the rails and chairs, as used in the New South Wales Railways, of which Mr. Fowler is the Consulting Engineer.

The sleepers are of colonial hard-woods, chiefly of iron-bark timber, rectangular in section, 10 inches wide, and 5 inches deep, and 10 feet in length. They are placed at an average distance of 3 feet apart from centre to centre,—being, for 21 feet rails, 3 feet 1 inch apart, but, at the joints, only 2 feet 6 inches, and for 18 feet rails, 3 feet 1 inch apart, but, at the joints, 2 feet 7 inches only.

The double-headed rails, Figs. 64 and 65, are of steel, $5\frac{1}{4}$ inches in depth, $2\frac{1}{2}$ inches wide, at the upper and lower tables, or heads, and $\frac{5}{8}$ inch thick at the web, or vertical portion. The upper and lower surfaces—the rolling surfaces—are curved in section to a radius of $5\frac{1}{4}$ inches, the depth of the rail. The sides of the head are rounded to a radius of ·59 inch, making a diameter, or thickness vertically, of $1\frac{3}{16}$ inches. The shoulders or underhangs of the tables are inclined to a slope of about 1 in 2, forming straight and equally-inclined bearings to receive the fishplates. The fish-plates, or splices, with which the ends of the rails are connected, are of steel, to the section shown in Fig. 64, and

17½ inches in length. They are ½ inch thick, slightly

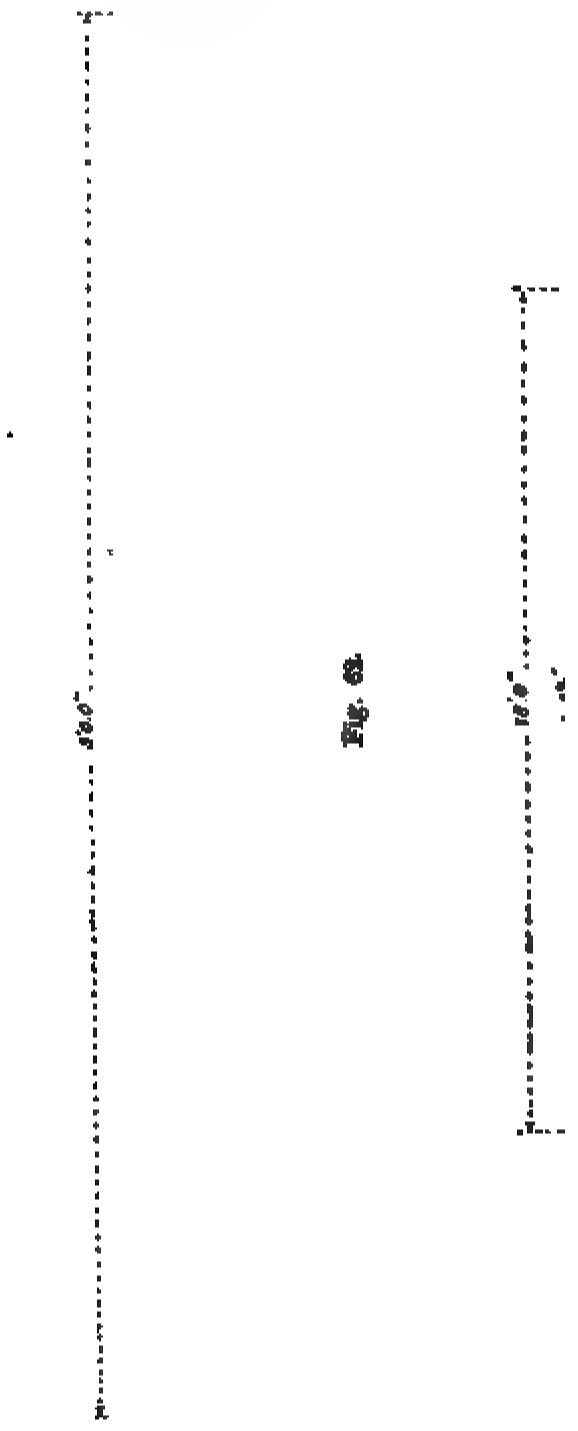


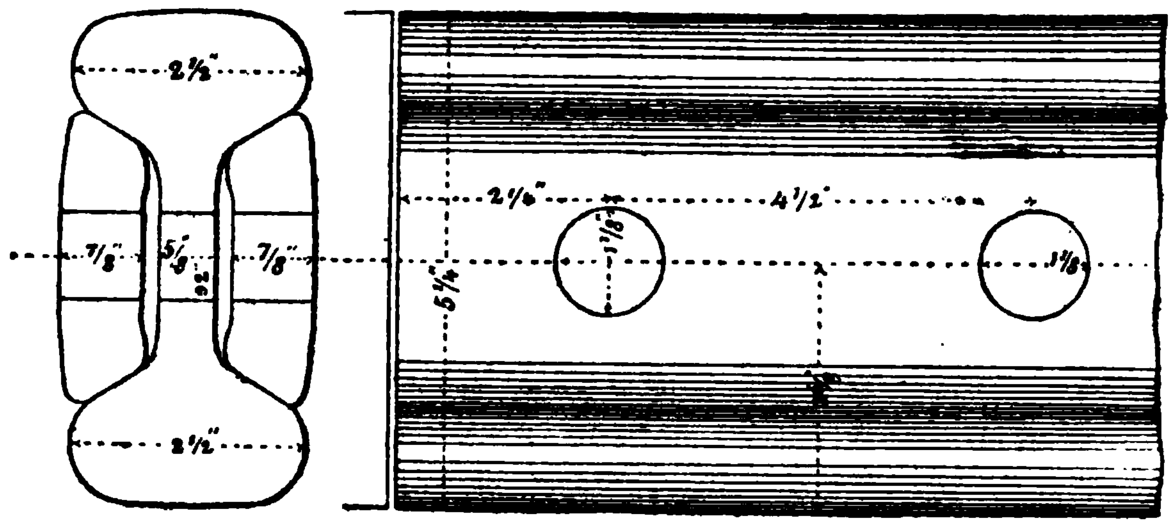
Fig. 63.

Fig. 63.

Type-Sections of Permanent Way, by Mr. John Fowler.

arched, and bevelled to fit the underhangs of the rails. They are applied in pairs, one plate on each side of the

rails, as shown, and are fastened with four $\frac{7}{8}$-inch bolts and nuts. The holes for the bolts, in the fishplates, are spaced apart to a pitch of $4\frac{1}{2}$ inches, so that the distance apart of the extreme holes amounts to ($4\frac{1}{2} \times 3 =$) $13\frac{1}{2}$ inches between their centres. The rails are laid with a clearance of $\frac{1}{16}$ inch



Figs. 64, 65.—Double-headed Rails.

apart between the ends, making the distance of the first hole in each end ($2\frac{1}{4} - \frac{1}{32}$) $2\frac{7}{32}$ inch from the end of the rail to the centre of the hole. The holes in the rails are drilled to a diameter of $1\frac{1}{8}$ inches, whilst those in the fishplates are $\frac{15}{16}$-inch in diameter, or $\frac{1}{16}$ inch larger than the bolts. The following is an extract from the specification for the double-headed rails, which are of steel :—

"The section of the rail is shown in Fig. 64, the weight being about 76 lbs. per yard; a template must be made by the manufacturers from the drawing attached, and must be approved by the Consulting Engineer, Mr. John Fowler, before commencing to roll. No rails weighing less than $75\frac{1}{2}$ lbs. per yard will be accepted, nor will any allowance be made for weight beyond $76\frac{1}{2}$ lbs. per yard. Within these limits the rails to be paid for at their actual weight.

"The length of the rails to be as under, namely :—

50	per cent.,	24 feet long.
40	,,	21 ,, ,,
10	,,	18 ,, ,,

and no deviation from these lengths exceeding $\frac{3}{16}$ inch will be allowed.

"The rails are to have ten holes drilled at each end of the rail for fishing, the exact positions and size of which are shown on the drawing. Any variation therefrom of more than $\frac{1}{32}$ inch subjects the rail to rejection.

"Each rail is to be marked on the side with the maker's name, year and month of manufacture, the initials N.S.W.G., and the word "Steel." None of these letters or marks to exceed $\frac{5}{8}$ inch in depth, so as to avoid interference with the jaws of the chairs.

"The ingots from which the rails are made are to be cast of the best steel for the purpose; the proportion of carbon to be used in combination is to be left to the discretion of the maker; but it must be such as to insure the rails being hard and tough, and capable of enduring the tests, as hereafter enumerated, without signs of fracture or deterioration. The rails must be made of uniform quality from good sound ingots, well hammered and reduced in cogging rolls, reheated, and then finished in the rolls while yet visibly red by daylight.

"*Conditions.*—Before commencing to roll the rails, samples must be sent to the Engineer, of the quality of the steel the Contractor proposes and will guarantee to use, and under no circumstances will he be permitted to sublet any portion of the contract, or to make the rails at any other works than his own, without the written consent of the Engineer.

"The rails to be of uniform section throughout, to be free from all defects, the ends sawn off true and square, free from roughness at the edges, and the straightening must be done without hammering. The surface of the rails to be entirely free from defects.

"The tests to be applied are as follows:—

"Certain rails of each day's make shall be selected, and each rail tested shall have a portion, 4 feet 6 inches long,

cut off. This shall be placed with the head upwards upon iron supports 3 feet 6 inches apart in clear, bedded upon a solid foundation.

"The rail to be then subjected to three blows from a weight of 1,800 lbs. falling 6 feet each time.

"The rail is to stand this test without showing any signs of fracture, or greater permanent set than 4 inches at the centre.

"One or more crop ends, or cut rails, shall be selected, which shall, if necessary, be cut to a length of about 2 feet. These shall be placed on the side, between two anvils 16 inches apart. They shall then receive one blow of a steam hammer (in the centre, by the intervention of a mandrill) capable of bending them from $2\frac{1}{2}$ to 3 inches; and shall afterwards be held on an anvil so as then to receive one or more blows of a steam hammer on the end, which shall bend them to not less than a right angle (the inner radius being not more than 4 inches), and in that condition they shall exhibit no signs of fracture or serious injury.

"The rails shall also be subjected to any other reasonable tests which may be directed by the Engineer, to ensure the rails being of the highest quality for strength and endurance."

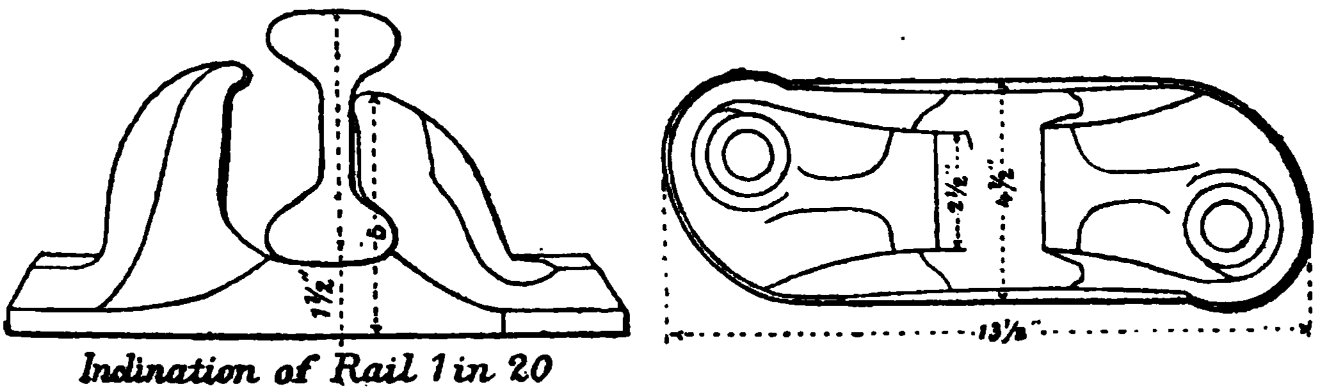
The fishbolts, as well as the spikes for fastening the chairs, "are all to be made from the finest quality of close fibrous iron. The bars from which the fastenings are made will be tested by bending, when cold, to an angle of 45° out of the straight line; they are then to be re-straightened, and after this test they shall show no signs of fracture.

"The fishbolts are to have cupped heads forged out of the solid: they are to be formed at the neck as shown, to prevent their turning round while being screwed up.

"They are to be $\frac{7}{8}$ inch in diameter, and all bolts which vary more than $\frac{1}{32}$ inch from the specified diameter, will be rejected. The manufacturer must provide himself with

some of the approved fishplates, and daily try the bolts to see that they fit properly in the plates. The screwed portion of the bolts to be of the exact length shown, the threads to be of Whitworth's standard."

The cast-iron chairs in which the double-headed rails are fixed are shown in Figs. 66, in which the rail is shown in its place, canted to an angle of 1 in 20 with the perpen-



Figs. 66.—Railway Chair.

dicular; and fixed with a hard-wood key or wedge. The chairs weigh 26 lbs. each. The sole is 13½ inches long, 4½ inches wide, and 1½ inches thick under the rail. Each chair is fixed to the sleeper by two spikes. The holes for the spikes are cast conically to a diameter of $\frac{7}{8}$ inch at the bottom, and $\frac{15}{16}$ inch at the top, and are afterwards cleaned out with a rymer to a diameter of $\frac{31}{32}$ inch at the top, tapering to $\frac{29}{32}$ inch at the bottom. The rail is fixed in the chair by means of a hard-wood key $1\frac{7}{8}$ inches thick. Test-bars of the metal used for casting the chairs, are cast to a scantling of 2 inches by 1 inch, and to 3½ feet in length. They are placed on edge, on supports 3 feet apart, and are required to carry a dead weight of 30 cwts. suspended from the centre of the bar, without fracture.

The wrought-iron spikes for the chairs are $\frac{7}{8}$ inch in diameter, with semi-spherical cup-heads 1½ inches in diameter, forged out of the solid. The neck of the spike is 1½ inches long, and is slightly taper, being $\frac{15}{16}$ inch in diameter next the head. The end of the spike is chamfered. The weight of the spike is about 1¼ lbs.

The above way, as laid in New South Wales, is bedded in ballast consisting of broken stone, 12 inches in depth below the sleepers, broken to a gauge of 3 inches; boxed up with broken stone of a smaller size, to a gauge of 2 inches, for a depth of 8 inches. The total depth of the ballast from the crown of the formation is 22 inches. The surface of the formation is slightly rounded in cross-section in order to drain off water penetrating through the ballast.

Mr. Fowler's standard model of permanent way on the system of single-headed flanged rails is illustrated by Fig. 67, for a single line of way. The sleepers are the same

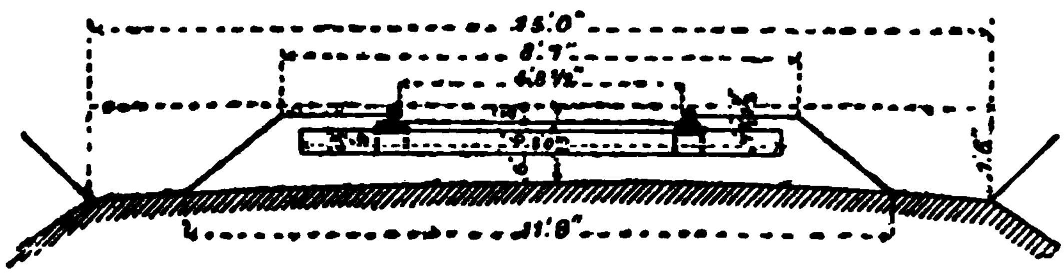


Fig. 67.—Type-Section of Permanent Way, by Mr. John Fowler.

in dimensions and in arrangement as those already described for the double-headed rail, except that they are only 8 feet in length. The inward cant of the rails, which is provided by the form of the chair for double-headed rails, is here provided by planing out by machinery the beds of the rails at the upper side of the sleepers, to the angle of 1 in 20; and that the rails may be kept in gauge, the beds are notched into the surface, to the thickness of the flanges of the rails.

The rails, Figs. 68 and 69, are $4\frac{3}{4}$ inches in depth, and $4\frac{3}{4}$ inches wide at the flange or base. The head is $2\frac{1}{4}$ inches wide, rounded at the upper surface, to a radius of 5 inches. The sides of the head are flat, and rounded into the upper surface, with a radius of $\frac{3}{8}$ inch. The web is $\frac{9}{16}$ inch thick, and is united to the head and flange by slopes of about 1 in 2, to give bearings for the fishplates. According to the specification, the weight of the rails was to be about $71\frac{1}{2}$ lbs. per yard; and it was to be within the limits of 70 lbs. and 72 lbs.

Two 1⅛-inch holes are drilled through the end of each rail; the holes are 4½ inches apart between centres, and the nearest hole is 2⅜ inches from the end of the rail to the centre. The rails are laid with ¼ inch clearance between the ends, so that when united, the nearest holes are 5 inches between centres; and adding twice 4½ inches, or 9 inches, the extreme distance apart of the bolt-holes amounts to 14 inches. No holes of any kind, either punched or drilled, were to be made in the flanges of the rails. The rails are fastened to the sleepers by screws and spikes alternately, having projecting heads, by which the flange is clipped and held down.

The tests applied in the course of manufacture of the rails were specified as follows:—

"Certain rails of each day's make shall be selected, and

Figs. 68, 69.—Single-head Flange-rail.

each rail tested shall have a portion, 4 feet 6 inches long, cut off; this shall be placed with the head upwards, upon iron supports, 3 feet 6 inches apart in the clear, bedded upon a solid foundation.

"The rail is to be then subjected to three blows from a weight of 1 ton falling 6 feet each time.

"The rail is to stand this test without deflecting more than 3 inches, and without showing any sign of fracture; and it is then to be placed under a steam hammer upon iron supports with the head upwards, and shall be further bent

by repeated blows to 8 inches out of a straight line in a length of 3 feet 6 inches, and must then exhibit no signs of fracture.

"The rails shall also be subjected to a suspended load of 25 tons between 3 feet 6 inches clear bearings, without permanent set exceeding $\frac{3}{16}$ inch, and to any other reasonable tests which may be directed by the Engineer, to ensure the rails being of the highest quality for strength and toughness."

It is added that, "if more than 5 per cent. of the rails so tested fail to bear the tests satisfactorily, the entire batch from which the rails were selected shall be liable to absolute rejection at the discretion of the Engineer."

The fishplates, in section, Fig. 68, are of steel. "They are to be sawn off square at the ends, and are to be perfectly true and fair in surface after cutting and punching, without any burr on the edges, or bulging, and to effect this, they are to be pressed while hot, in a press provided with dies properly contoured to the section of the plates." Each plate has four holes punched in it, $\frac{15}{16}$ inch wide, and angled at each side, to receive the corresponding square angles on the necks of the bolts. The fishplates weigh about 22 lbs. per pair, they are, when finished, heated and dipped in linseed oil. The fishplates are like those which have already been described for the double-headed rails.

The screws and spikes for fastening the rails to the sleepers are manufactured of $\frac{3}{4}$-inch round iron, with the heads forged out of the solid. The screwed part of the wood-screw tapers $\frac{1}{8}$ inch in diameter. The screws and the spikes weigh 1 lb. each.

In the United States, the Vignoles, or flange-rail, is almost universally used for railways. It varies in weight from 67 lbs. or 70 lbs. per yard, on a few of the leading lines, to 30 lbs. on narrow-gauge lines. Fully 60 per cent. of the length is laid with rails of 56 lbs. per yard; and rails of 60 lbs. and 50 lbs. per yard are next in frequency. Captain

Galton describes the standard way or track of the Pennsylvania Railroad, shown in Fig. 70. It is constructed with flanged steel rails, of two sections, one of 60 lbs. per yard (Fig. 71), the other of 67 lbs., in lengths of 30 feet. The 60-lb. rail is 4¼ inches deep, and the 67-lb. rail is 4½ inches deep. The head of the 60-lb. rail is 1⅜ inches deep, and that of the 67-lb. rail is 1½ inches deep. The fishes or splices are 2 feet in length; they are held by four bolts, two on each side of the joint. The outer splice is formed with a horizontal flange or "tongue," which overhangs the flange of the rail, and is spiked to the sleeper. The joint is "suspended" between two sleepers. Allowance for expansion, when the rails are laid in winter, is provided by leaving a space $\frac{5}{16}$ inch clear width between the ends of the rails; in summer, a space only $\frac{1}{16}$ inch wide is allowed. The sleepers are 8 inches wide by 7 inches deep, and 8½ feet in length. They are laid so closely that the maximum distance apart between centres does not exceed 2 feet. There are 16 sleepers for each length of 30 feet, and the sleepers at the joints are laid with a clearance of only 10 inches between them. The rails are laid to a gauge of 4 feet 9 inches; they are fastened by spikes to each sleeper, at the inside and the outside. The width for the double line of way at the formation level is 31 feet 4 inches in cutting. On embankments, the width of the formation is 24 feet 3 inches. The surface is formed with a slope of 1 in 20 from the centre to each side. The ballast is laid to a depth of not less than 12 inches under the sleeper, and is filled up to the level of the upper

Fig. 70.—Permanent Way, Pennsylvania Railroad.

Fig. 71.—Fish Joint.

surface of the sleepers. At the outer edges, it is sloped down to the formation level. Where stone ballast is used, it is broken uniformly to a gauge of 2½ inches in diameter. For double lines, large stones are placed in the bottom, at the centre between the lines, to provide for drainage; but those stones are kept apart from the ends of the sleepers. Thus water is drained off rapidly.]

CHAPTER X.

METALLIC PERMANENT WAY OF RAILWAYS.

[METALLIC permanent way, in which the sleepers are of iron, has been much employed in tropical countries, and is now to some extent adopted in France and in Germany. The oldest and most widely used system of metallic way is that of Mr. H. Greaves, who, in 1846, introduced a spherical or bowl sleeper of cast-iron, having the chair for the rail cast on its apex. The alternate sleepers are connected and kept to gauge by transverse bars, which pass through and are bolted to them. The form of the sleeper is strong, it holds well in the ground, the chair is not liable to be detached, the whole bearing surface is directly beneath the road, the ballast is kept dry and elastic; and there is a simple means for packing the sleeper through the holes, with a pointed rammer, from the surface, so that the sleeper and the rail can be forced upwards without disturbing the general bed of ballast; or they may be lowered by taking out a portion from the interior.

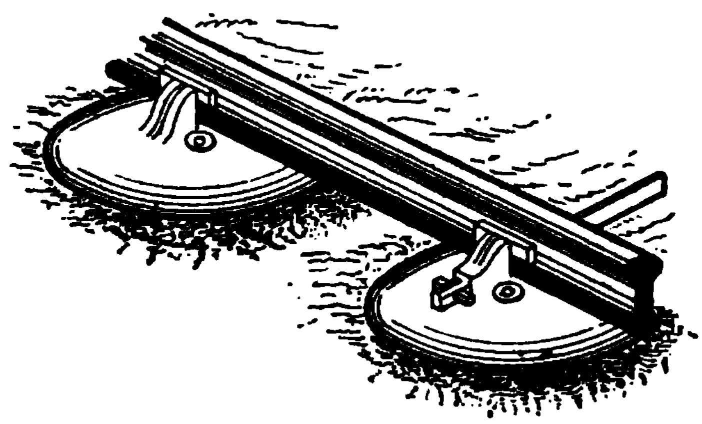


Fig. 72.—Bowl Sleepers.

Wrought-iron transverse sleepers were first tried in Belgium in 1862, then in France and in Portugal, and afterwards in Germany. One of the first of them was the Couillet sleeper, like I in section, 7 inches wide, with a

shallow flange at each edge, and about 8 feet long, weighing 100 lbs. It was laid flat on the ballast, and a flange-rail was bolted to it, bedded on a cushion of hard-wood. The Le Crenier sleeper was also shallow in section, like ⊓, consisting of plate-iron turned down at the edges. It was 12 inches wide, and 8 feet long, and the flange-rail, placed directly upon it, was fixed by brackets bolted to the sleeper. These and other wrought-iron sleepers were tried, and were found to be deficient in vertical stiffness, and inconvenient for the operations of packing, whilst they offered little resistance to lateral displacement of the way.

The Vautherin sleeper, first tried in 1864, on the Lyons Railway, has been successful. It is hollow in section, of the form A, truncated, supposing the upper part of the letter to be removed; presenting a flat bearing surface, 3¼ inches wide, for a flange-rail. It is 8 feet in length, and 9 inches wide, over the flanges forming the base. It is ⅜ inch thick at the centre, and is only about half that thickness in the wings. The rail is fixed to the sleeper with gibs and cotters. It has been reported that the motion over the Vautherin sleepers is much easier than that over sleepers of oak, and that in consequence the cost of maintenance is comparatively small. It is stated that among a number of rails, laid for trial under similar conditions, some of them on wooden sleepers, and some of them on Vautherin sleepers, the number of defective rails amounted to only 2½ per cent. of those laid on Vautherin sleepers, against 13 per cent. of those laid on wood. It was found that if the Vautherin sleepers were not at least 8 feet in length, they failed at the ends, and that even for this length it was expedient to strengthen them at the angles. It was also found that large and hard ballast, or broken stones or broken slag, aggravated the tendency to give way. Ballast of ashes produced a similar bad effect, and also caused the sleepers to rust. On the contrary, ballast of gravel, of a marly character, adapted itself admirably to the

form of the sleeper. The system of fastening the rails to the sleepers by gibs and cotters is being abandoned in favour of clips and hook-bolts.

The Hartwich system of iron way is an instance of heroic attempt at improvement. It consists of a flange-rail 9½ inches deep, having a web $\frac{7}{16}$ inch thick, and a foot 5 inches wide. The rail weighs 87 lbs. per lineal yard; it is fish-jointed, and is strengthened at the joint by a sole-plate 19 inches long, 9 inches wide, and ⅝ inch thick at the centre, secured to the rails by eight bolts and nuts connected transversely by two sets, at different levels, of screwed tie-rods. The rails are laid in trenches filled with gravel and small stones. This system has given bad results wherever it has been laid. The rails soon became curved longitudinally. On one line, after 3¾ years of service, they acquired a permanent set of 1 inch, and had to be taken up. The renewals were frequent and troublesome.

The Hilf system of iron way consists of two parts: an iron longitudinal sleeper, and a flange-rail of steel. It is simple, easily laid and maintained, and economical. The sleeper is in section like the letter E, bevelled at the angles, having an upper flat surface, and three flanges downwards. It is 12 inches wide, and about 2½ inches deep, and it can be rolled to lengths of 30 feet and only ⅓ inch in thickness, and to a weight of 59 lbs. per yard. The rail is 4·32 inches high, with 2·32 inches width of table, 3·40 inches width of flange-base, and $\frac{4}{10}$ inch thickness of web. It is rolled in lengths of 30 feet, and weighs 51½ lbs. per yard. It is fish-jointed, and is fixed to the sleeper with two rows of bolts and nuts at intervals of from 30 to 40 inches. The gauge is preserved by means of 1-inch tie-rods, screwed at both ends, with nuts. One tie-rod is sufficient for each length of rail. The combined rail and sleeper, placed on supports 54 inches apart, can carry 18 tons at their middle, without injuring their elasticity.]

CHAPTER XI.

RAILWAY SWITCHES AND CROSSINGS.

It is frequently necessary to pass trains from one line of rails to another, and several different methods have been devised for doing this. One of the simplest and most frequently-adopted plans is to lay down a short line of rails, connecting the other two, and so establishing the desired communication. It becomes necessary, however, then, to have the means of connecting and disconnecting this short with the main line at pleasure, according as it is intended that the train should leave or continue upon the latter; and this is effected by means of a contrivance termed a *switch*, which is shown in Fig. 73: *a b* and *c d* are portions of the rails of the main line, and *e f* and *g h* portions of the short line branching from it, all of which are immovably fixed in the ordinary manner, with the exception of the two rails *f i* and *k l*; these, which are termed the *tongues* of the switch, are only fixed at one of their ends, *f* and *k*, on which they turn as centres; their other ends are tapered away to nearly a point, a slight recess being cut in the other lines, at *i* and *l*, into which they fit. These tongues are connected together by a bar, *m n o*, by means of which they are always preserved at such a distance apart, that when either of the tongues is in contact with the rail near it, the other shall be removed from the opposite rail sufficiently to leave space for the flange of the carriage-wheels to pass between them. In order, then, to cause a train to pursue its course along the

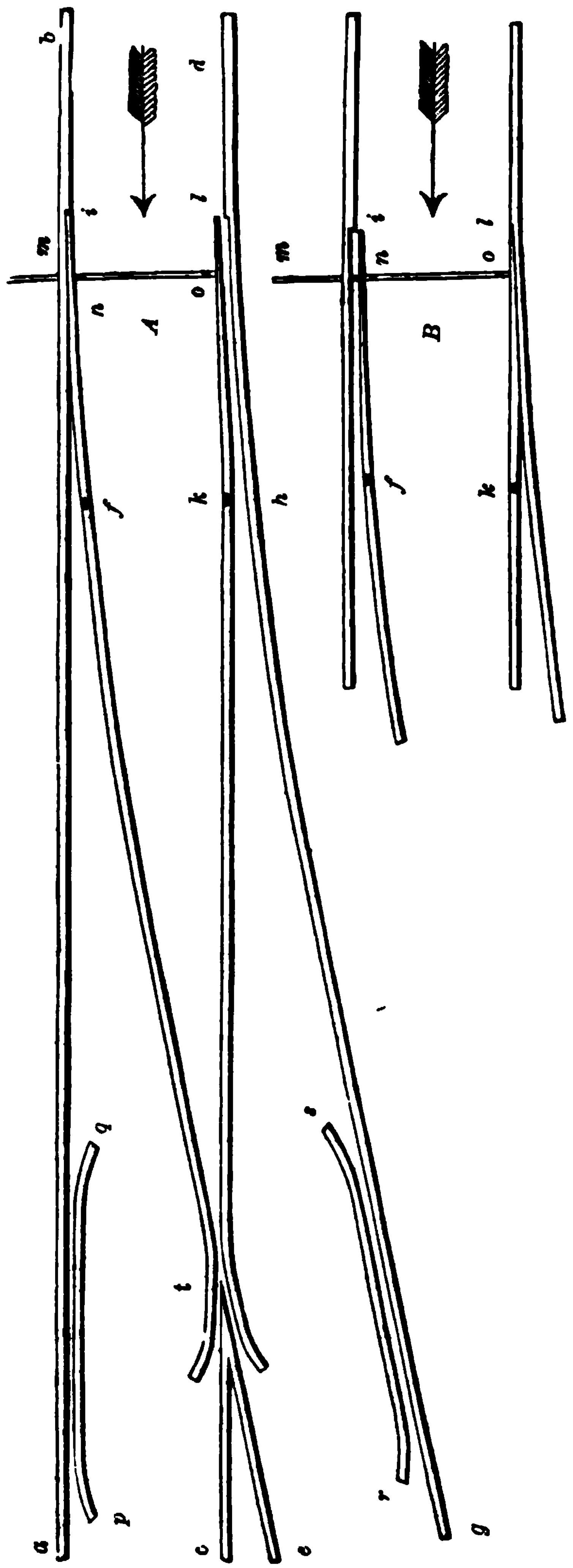


Fig. 73.—Points and Crossings.

main line, or to leave the same and enter the branch line, it only becomes necessary to move the bar *m n o*, which, when in the position shown at A, will cause the carriages to leave the main line, but if shifted into the position shown at B, will cause them to continue their course along the same. It is usual to have the switches so arranged that they are kept in the position shown in B (in which the main line is not interrupted) by a self-acting weight, the attendance of a man to move them into the position shown at A being necessary when it is desired that the train should leave the main line. Two guard rails, *p q*, *r s*, are necessary in order to prevent the flanges of the wheels from striking against the point where the two lines intersect each other.

Another method of removing only single carriages from one line of rails to another, is by means of what is termed a *turntable*. Three of these are shown in Fig. 74, at A, B, and

Fig. 74.—Turntables.

C. They consist of a circular platform of timber or iron, supported on wheels, and fixed upon a centre in such a manner that it is capable of being turned round, even when loaded with a considerable weight, without much friction. On their upper surface they have usually two lines of rails crossing at right angles, and they are so placed that these form the continuation of the main lines of the railway, and another line crossing these at right angles, as shown in

the figure. Now, the way in which these are employed is as follows: supposing that a number of carriages situated on the line F C were required to be removed on to the line D A, the carriage nearest C would be moved on to the turntable C, (which, it should have been stated, is of sufficient diameter to receive the whole of the carriage upon it,) and brought into the position shown by the whole lines, *a b c d*; the turntable would then be turned upon its centre through a quarter of a circle, by which the carriage would be brought into the position shown by the dotted lines *e f g h*; it would then be run over the turntable B, on to A, into the position shown by *i k l m*, and the turntable A being turned upon its centre, would bring the carriage into the position shown by the whole lines, *n o p q*, in which it would only have to move down the line of rails to D; and the same method of procedure being followed with the other carriages, the whole train would in a very short time be shifted from one line to the other. If it had been desired to bring the carriage on to the centre line of rails, then the turntable B would have been employed instead of A.

A simple method of reversing a train of carriages is shown in Fig. 75, which consists in forming a short branch, E F, at right angles with the main line, and communicating with it by two curves, B E and E C. The train has only then to be run off the main line, by the curve B E, into the branch, until the last carriage has cleared the point E, when the switches are altered, and the train returned to the main line by the other curve, E C, by which the whole train will have been reversed, the end which before was towards A being now towards D.

[Switches constructed of ordinary double-headed rails are open to the objection of the insufficient wearing surface of the lower table of the rail on the chair, and to the instability arising from the great height of the tongue rail, relatively to

the width of the base. Hence the adoption of specially-formed rails for the construction of the points. Wild and Parsons' switch, of which the section and contour lines are

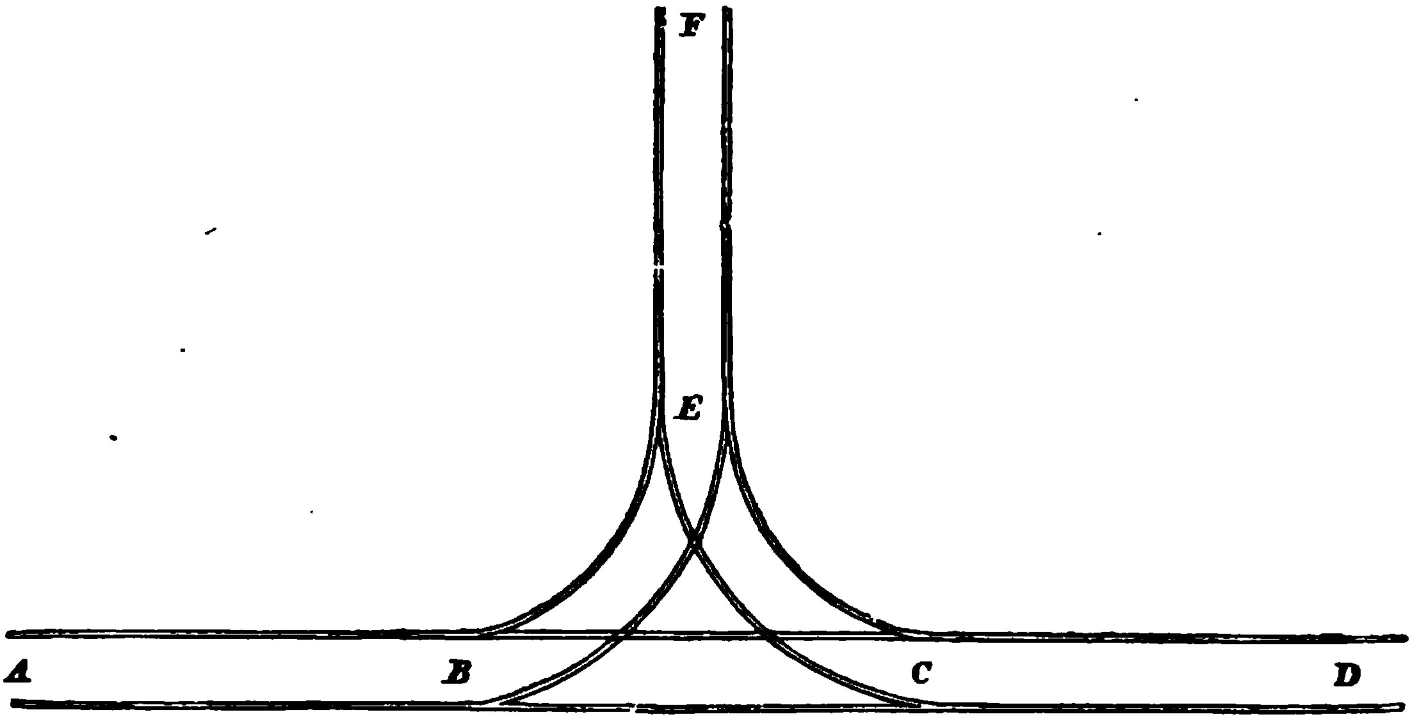
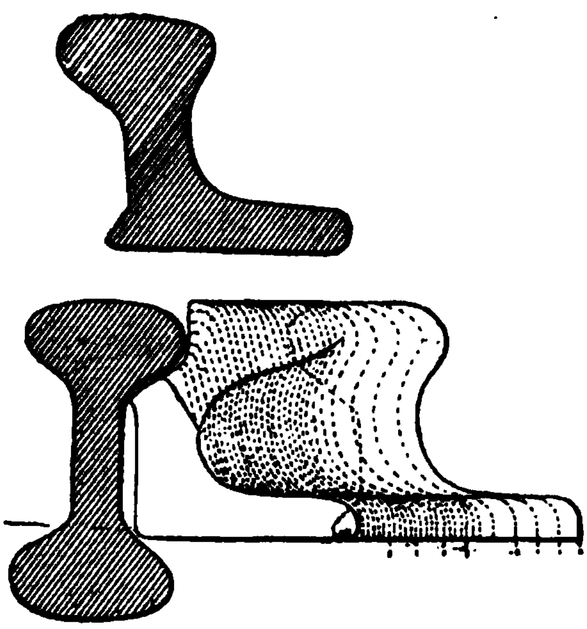


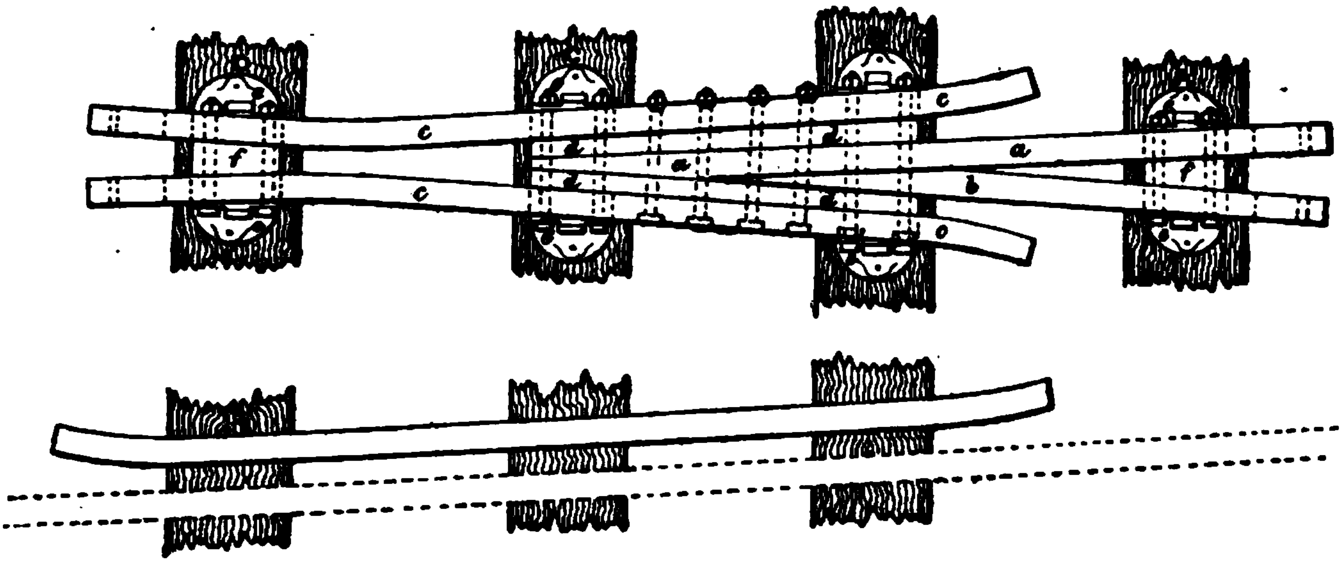
Fig. 75.—Reversing Curves.

shown in Figs. 76, is on a good model. The tongue rail, Fig. 77, has great width of base—nearly 4 inches—whilst the height is only 3¾ inches, as against 5 inches, the height of the fixed rail. It thus possesses both stability and transverse strength. Again, the tongue is so tapered that the end of it is housed under, instead of being notched into, the upper table of the fixed rail; so that the train may be transferred without shock from the fixed line of rails to the siding, or *vice versâ*. The fixed rails are secured in the chairs by wedges instead of pins as formerly. The tongue rails are each 12 feet long. There are two heel chairs, to which the switches or tongue rails are pinned, on which they turn, and eight intermediate chairs, on which they slide.



Figs. 76.—Wild and Parsons' Switch.

Parsons' reversible steel crossing, Figs. 77, is constructed of steel rails, similar in section to an ordinary double-headed rail, except that the upper table is formed square at one



Figs. 77.—Parsons' Crossing.

side. This formation adds to the bearing surface of the rail where it is most needed—at the gap—the wheels being thus well supported by the wing-rails before quitting the point, and passing smoothly over the gap.]

CHAPTER XII.

TRAMWAYS.

[A TRAMWAY, in the modern sense of the word, is a street-railway, or a road-railway, forming part of the road or the street, and constituting, with the carriage-way, a combination of railways and common thoroughfares, such that the traffic of the street or the road, unaffected by the tramway, is free to circulate. It follows, as the principal condition of such free circulation, that the surface of the rails, whilst these are adapted for carrying flanged wheels, should be substantially at the general level of the carriage-way.

The modern tramway was first employed in the United States, where it was urgently wanted, in consequence of the inferior condition of the streets and roads of the large cities. The first American tramway was the New York and Haarlem line, of which the first section, laid in the main thoroughfares, was opened in 1832. It was laid to a gauge of 4 feet 8½ inches. But it was unpopular, and was for a time suppressed. Tramways, nevertheless, were revived in the same city, about the year 1852, by the instrumentality of M. Loûbat, a French engineer, who recommended and laid down a tramway consisting of rolled wrought-iron rails laid upon wooden sleepers. The rails were constructed with a groove in the upper surface, to guide the wheels of the cars, which were made with flanges, like those of railway carriages and waggons. Tramways were rapidly multiplied in New York, which owes much of its development to the tramways, the

traffic on which is of much more importance than that of the light-wheeled vehicles used for ordinary circulation; otherwise the rails, which were formed with wide, gulf-like grooves, would not have been tolerated in the streets. The tramway afforded incalculable advantages, and it became an indispensable feature in the principal cities of the United States. The long distances to be traversed, the generally bad condition of the streets and roads, and the comparative scarcity of other vehicles, formed a combination of circumstances which forced the tramway-car into general use for all classes.

Habits were formed, and the irregularities of rails and roads were of less importance than they had been felt to be in England. The annexed sectional illustrations, Figs. 78, of

Figs. 78.—Tram-rails, New York.

tram-rails in New York, shows the fearless manner in which New York tram-rails were proportioned—combining ob-

noxious grooves with massive sections. An unsophisticated observer, struck by the proportions of the rails in New York with their portentous grooves, described them as "rails which have a sort of iron gutter attached to each on their inside edge." The following are the leading particulars of some tram-rails in New York:—

TRAM-RAILS IN NEW YORK.

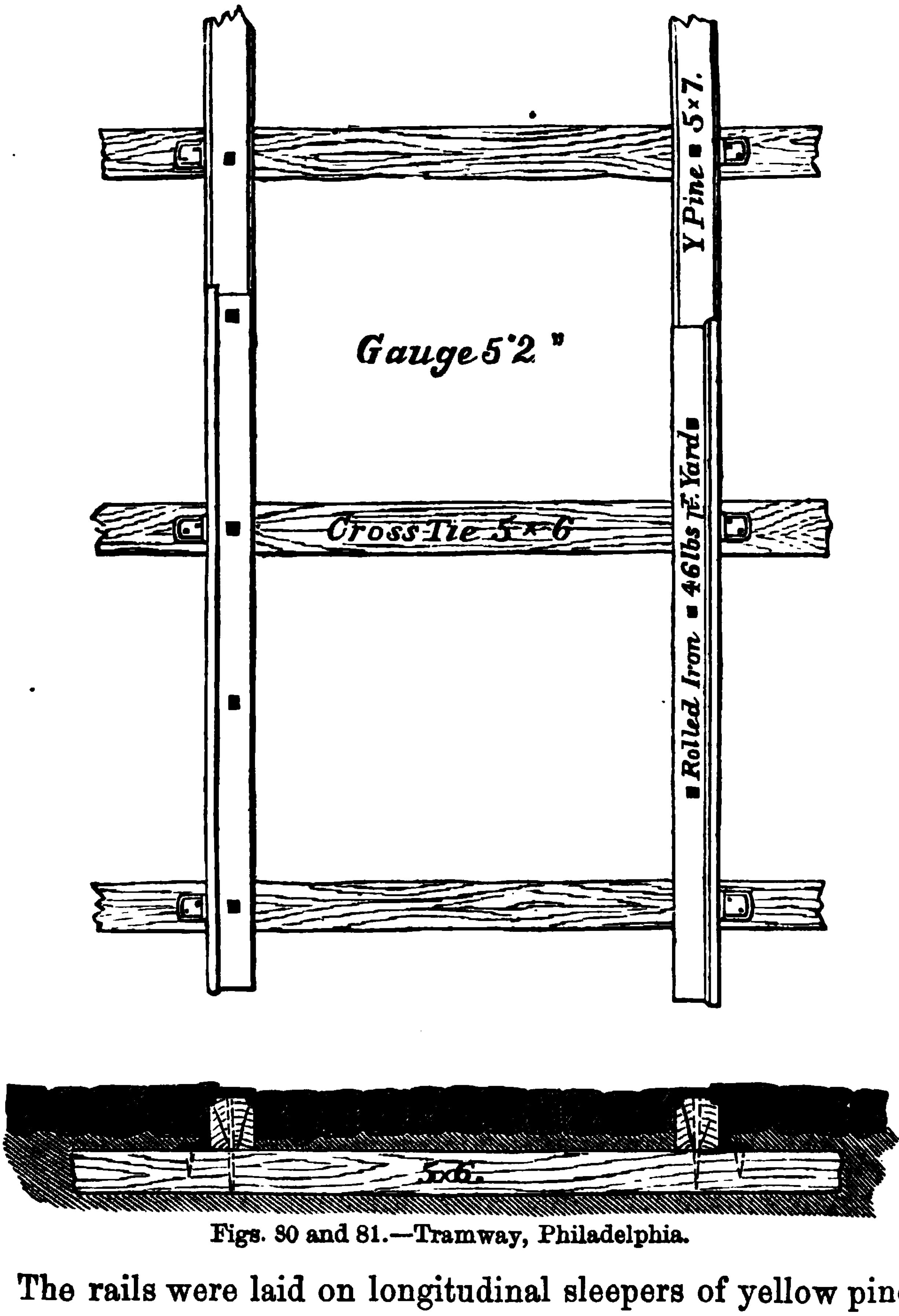
Tram-rails.	Weight per yard.	Depth of Groove.	Depth at Head.	Total Width.
	lbs.	Inches.	Inches.	Inches.
New York and Haarlem	98	$1\frac{5}{8}$	$2\frac{1}{4}$	$5\frac{3}{4}$
Brooklyn City	67	$1\frac{1}{4}$	$1\frac{13}{16}$	$4\frac{7}{8}$
New York, Second Avenue ..	69	$1\frac{1}{4}$	$1\frac{11}{16}$	5
" " Third " ..	90	$1\frac{1}{4}$	$2\frac{3}{8}$	$5\frac{5}{8}$
" " Sixth " ..	76	$1\frac{5}{8}$	$2\frac{1}{4}$	5
" " Eighth " ..	61	1	$1\frac{1}{2}$	5

In order to mitigate the inconveniences of the New York sections of tram-rails, a different form of rail—a "step-rail," as it may be called, Fig. 79, from which the groove was

Fig. 79.—Tram-rail, Philadelphia.

banished though a ridge remained—was introduced in Philadelphia, and laid in Fifth and Sixth Streets, where it gave satisfaction. It consisted of a flat plate, 5 inches wide, formed with a raised ledge or fillet at one edge, standing $\frac{7}{8}$ inch above the surface of the plate, without any groove. The plate was formed with a ledge or fillet at each side, below, let into corresponding rebates in the upper corners of the

sleepers. The weight was 46 lbs. per yard. The gauge was fixed at 5 feet 2 inches between the ledges, to suit the wheels of ordinary vehicles, which could run on the lower flat surface. The type of tramway thus settled for Philadelphia, in 1855, is shown in Figs. 80 and 81.



Figs. 80 and 81.—Tramway, Philadelphia.

The rails were laid on longitudinal sleepers of yellow pine, 5 inches wide and 7 inches deep, bolted down upon transverse

sleepers, 6 inches wide and 5 inches deep, with iron knees to maintain the rails in gauge.

The step-rail is in general use in the principal cities of the United States, where probably there is less of the light cab and omnibus traffic than prevails in English cities exposed to the action of the obnoxious step. The gauge of tramways, adopted for the most part in the United States, is 4 feet 8½ inches.

The modern tramway was introduced in England by Mr. G. F. Train, who, in 1857, made proposals for laying tramways, on the system originated in Philadelphia, in some of the metropolitan thoroughfares and in a few provincial towns. Lines were laid in a few places; but, after brief periods of trial, the lines were removed, though in some places, as at Birkenhead, flat-grooved rails were substituted for the step rails.

In 1866 and 1867, application was made to Parliament for power to construct a system of tramways in Liverpool, for which an Act was obtained in 1868. This was the first English system of tramways for passenger traffic that was authorised by Act of Parliament. The works were constructed under Mr. George Hopkins, as engineer-in-chief, to a gauge of 4 feet 8½ inches. The form of the rails adopted in the original construction of the Liverpool tramways was of a flat-grooved section, such as had been found to answer satisfactorily at Birkenhead, though narrower, weighing 40 lbs. per yard, about 1 inch in thickness, and having a section area of about 4 square inches. Rails of similar but larger section were afterwards employed, weighing 45 lbs. per yard, shown in Fig. 82. The rail was little else than a flat bar, having a narrow and shallow groove in its upper surface, with a fillet on lower side, and bedded on a longitudinal sleeper. The rail was 4 inches wide, and 1⅜ inches in thickness. The groove was formed with sloping sides, and was ¾ inch in depth, with a width of ⅝ inch at the

bottom, and double the width at the surface of the rail. The tread, or rolling surface for the wheels, had a width of about 2 inches, when, of course, the inner edge of the tread

Fig. 82.—Early Tram-rail, Liverpool.

was at the half-width of the rail; whilst the ledge forming the other side of the groove was about $\frac{7}{8}$ inch wide at the surface, and was corrugated transversely to prevent slipperiness for horses. The rails were bedded on timber sleepers, 4 inches wide and 6 inches deep, and were fished with $\frac{7}{8}$-inch wrought-iron plates, 12 inches long and 4 inches wide, applied below the joint, let flush into the upper side of the sleeper. The joint was fixed with four vertical spikes, two to each rail, driven through the rails, at the bottom of the groove and the fish-plate, into the sleeper. The rails were also spiked at intervals to the sleepers. The heads of the spikes were countersunk and let into the rails to finish flush with the bottom of the groove. The combined sleeper and rail thus presented, for the most part, a vertical surface at each side, against which paving-stones could be closely and evenly laid and jointed. The construction of the way is shown by Figs. 83, 84, 85. To render the way independent for support, on uncertain or on broken ground, the roadway was excavated to a depth of 14¼ inches for the whole width, and a continuous bed of lime concrete, 7 inches thick, was laid for the whole width of the track, as a foundation, upon which the sleepers were placed. The interspaces between the sleepers were filled up with cement to the right level for supporting 4-inch cubes. The sleepers

were laid in and spiked to cast-iron clip chairs, Figs. 84 and 85, which were placed about 4 feet apart longitudinally, and rested direct on the concrete foundation. The gauge of the rails was fixed by bar-iron cross ties, 1½ inches deep by ⅜ inch thick, the ends of which were dovetailed into grooves cast in the inner sides of the chairs. The chairs were 6 inches wide at the joints of the sleepers, and 8 inches intermediately. The roadway was nearly all of macadam, and the materials for the concrete were taken from the macadam which was lifted to make room for the line; whilst the whole of the surface between the rails, and for a width of 18 inches beyond the outer sides of the rails, was paved with Welsh granite sets—4-inch cubes between the rails, and sets of 6 inches in depth for the outer 18-inch spaces. The outer width, 18 inches, was provided in the Act, and it defined the marginal boundaries of the breadth of roadway to be maintained by the tramway company. That width was, and is now, accepted as a fair compromise; and, says Mr. J. Morris, "it does fairly represent the extent of possible injury even which

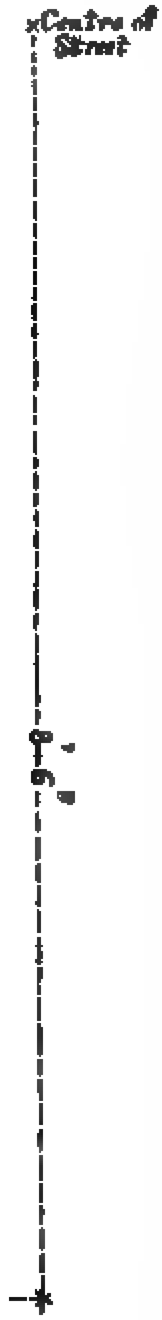


Fig. 83.—Early Tramway, Liverpool.

the tramway can do to the road, and it is accepted universally on the Continent, and almost universally in America, and is the recognised standard." *

Fig. 84.—Tramway, Liverpool.

The tramways of Constantinople, of which M. Lebout was the engineer, were constructed with the pattern of grooved rail, weighing 46 lbs. per yard, employed in the Paris tramways, fastened as in Fig. 86. The rails were bolted to longitudinal sleepers, as in Fig. 87, laid on a bed of sand 6 inches deep, spread on the bottom of the excavation. The

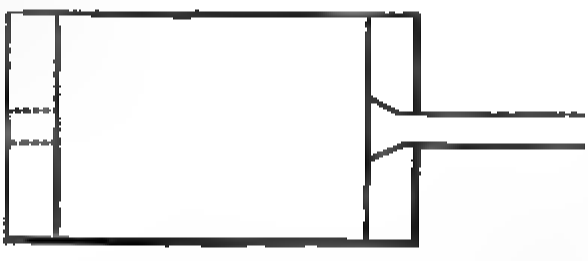


Fig. 85.—Plan of Chair and Tie.

Fig 86.—Tramway, Constantinople.

longitudinal sleepers were connected by round iron tie-rods,

* *Report of the Select Committee on Tramways Bill*, 1870.

which were passed through them, and were screwed up by nuts at both sides of the sleepers, as shown in the section of the way, Fig. 86. Streets in Constantinople, of from 13 feet to 23 feet wide, were paved across the whole of the way, as shown in Fig. 88.

Fig. 87.—Section of Tramway, Constantinople.

The total lengths of streets traversed by tramways, in the United Kingdom, on June 30, 1876, were as follows:—

	Miles.
England and Wales	132·22
Scotland	41·30
Ireland	25·09
Total	198·61

The section of tramway employed by Mr. G. Hopkins in the reconstruction of the North Metropolitan Tramways, in 1877, is shown in Fig. 89. The pre-existing foundation of concrete was partly renewed by the excavation, under each sleeper, of a shallow trough in the concrete, 1½ inches deep and 6 or 7 inches wide. This trough was filled with fine concrete, in which the longitudinal sleepers were embedded to a depth of half an inch. The sleepers are 4 inches wide and 5 inches deep, rebated at the upper side to fit to the rail. They are bedded at the joints on plates of fir, 8 inches wide and 2 inches thick, let into the foundation. The rails are of steel, weighing 60 lbs. per yard. They are

Fig. 88.—Tramways, Constantinople.

$3\frac{7}{8}$ inches in width at the surface, $2\frac{3}{4}$ inches deep over the flanges, and $1\frac{4}{16}$ inch thick. The groove is $1\frac{1}{4}$ inch wide and $\frac{3}{4}$ inch deep, leaving only $\frac{9}{16}$ inch of metal below the groove. The tread is 2 inches wide, and it is very slightly rounded. The flanges are $\frac{5}{8}$ inch thick at the edge. Each rail, of 24 feet in length, is fastened by 25 staples, placed at a pitch of 2 feet 7 inches at each side, except at the ends, where there are two pairs of staples.

Fig. 89.—North Metropolitan Tramway.

The Glasgow Corporation tramways, constructed in 1874-75, afford an excellent example of combined wood and iron for the way, Fig. 90. The ways were laid to a gauge of 4 feet $7\frac{3}{4}$ inches, with an interspace of 3 feet $11\frac{1}{2}$ inches between the two lines; whilst the paving was extended for a width of 18 inches at each outer side. The total width for a double line was made up thus—

	Feet.	Inches.
Two widths of gauge	9	$3\frac{1}{2}$
Interspace	3	$11\frac{1}{2}$
Two strips of pavement	3	0
Four half-widths of rail ($1\frac{7}{8}'' \times 4 =$) .	0	$7\frac{1}{2}$
	16	$10\frac{1}{2}$

For a double line, the roadway was excavated for a width of 17 feet, to a uniform depth of $12\frac{1}{4}$ inches below the intended level of the rails. The rails were of wrought iron, and weighed 60 lbs. per yard. They were rolled in lengths of 24 feet, with about 5 per cent. of the total quantity in shorter lengths. They are $3\frac{7}{8}$ inches wide and $1\frac{7}{16}$ inch thick. The rolling surface, which is slightly rounded, is $1\frac{7}{8}$ inches wide, the groove is $1\frac{1}{4}$ inches wide, and the flange at the inner side is $\frac{3}{4}$ inch wide. The groove is formed with a flat floor, and is only $\frac{11}{16}$ inch deep, having a $\frac{3}{4}$-inch thickness of metal below it. The longitudinal sleepers or beams are of Baltic

Fig. 90.—Glasgow Corporation Tramway.

red timber, 4 inches wide and 6 inches deep. Each 24-feet rail is fastened to the beams by 20 side staples. The transverse sleepers, under the longitudinals, are 8 feet long and 4 inches deep; 6 inches in width, except at the joints, where they are 7 inches wide. All the timber was creosoted to the extent of 10 lbs. of creosote per cubic foot. The longitudinal beams rest in cast-iron chairs spiked to the sleepers. The spaces between the sleepers were filled with concrete. The paving sets were laid on a $\frac{1}{2}$-inch layer of sand, and were grouted with a mixture of bitumen and pitch-oil.

The "inner circle" of the Liverpool tramways was relaid in 1877-8 on the system of Mr. G. F. Deacon, the borough

engineer at the time. The leading feature of the system, Fig. 91, is the method of fastening the rail to the longitudinal sleeper and the foundation of concrete by means of a central bolt. The groove is formed centrally in the rail, which affords a bearing over its whole width for wheels correspondingly formed, with a central flange. On the bottom, a foundation of concrete, made with Portland cement, 7 inches deep, was laid for the whole width of the street, and finished with a perfectly smooth surface. The longitudinal timber sleepers are 9¼ inches wide and 5⅞ inches deep. The rails are of Bessemer steel, weighing 61 lbs. per yard, rolled in lengths of 24 feet 2 inches, with shorter lengths. They are 3¼ inches wide, and 3¼ inches deep over the flanges. The groove is in the middle of the upper surface, 1 inch wide and $\frac{11}{16}$ inch deep. The upper bearing surfaces are each 1⅛ inches wide—together, 2¼ inches. The rails are bedded with coal tar on the sleepers, and are fastened by means of central ¾-inch bolts, each of which is formed with an eye at the upper end, which embraces a ¾-inch round iron cross pin, passed horizontally through holes in the side flanges of the rails. The bolt passes down through the sleeper, and nearly through the stratum of concrete, and is formed with a head at the lower end, which takes a bearing upon a round cast-iron plate or washer 6 inches in diameter, which, with the lower portion of bolt, is embedded in the concrete. The bolt is adjustable in length by means of a right-and-left handed double nut. The paving-sets are from 7 to 7½ inches

Fig. 91.—Liverpool Tramway.

deep, laid on a ½-inch bed of sand; except the sets next the rails, which consist of the most durable stone,—the hardest granite, or coarse-grained trap.

Mr. James Livesey, so early as in 1869, advocated the use of an iron substructure for tramways, combining a rail having side flanges, with cast-iron stools placed at intervals. As applied in the city of Buenos Ayres, two kinds of this system are shown in Figs. 92 to 95. The steel groove-rail, Figs. 92 to 94, was employed for the City lines. It weighed 40 lbs. per yard. It is 3½ inches wide. The supporting stools are 3 feet apart, fixed in couples on wrought-iron base plates. The rail is dovetailed

Fig. 92.—Livesey's Tramway.

Fig. 93.—Livesey's Way.

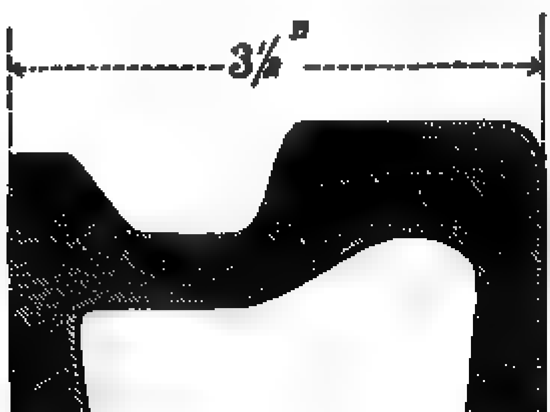


Fig. 94.—Livesey's Tram-rail.

Fig. 95.—Livesey's Tramway.

over the stool, to which it is keyed. The second kind of tramway, Fig. 95, used in the suburban districts of Buenos

Fig. 96.—W. J. Cockburn-Muir's Tramway.

Ayres, has a flanged or Vignoles rail, fixed by hook-bolts and nuts to cast-iron stools.

Fig. 97.—Block Sleeper.

Mr. W. J. Cockburn-Muir's system of iron way, which he calls the "block-sleeper system," in which the rail is supported on cast-iron stools or blocks, is shown in Figs. 96

to 98. The rails are of wrought iron, having a middle vertical web on the under side. They are 3 inches wide, and weigh 30 lbs. per yard. The sleepers are cast-iron blocks, rectangular, hollow, open at the base, and ribbed interiorly. They are about 11½ inches long, 7½ inches wide, and 6 inches deep; placed at 2 feet 6¼ inches apart between centres, and tied transversely. The system has been adopted for the tramways of Monte Video and elsewhere.

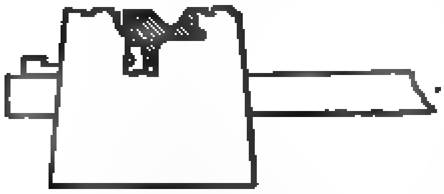


Fig. 98.—Block Sleeper.

Ransomes, Deas, and Rapier's system, Figs. 99, 100—a

Fig. 99.—Ransomes, Deas, and Rapier's Tramway.

cast-iron way laid on concrete—was laid in 1870 at Glasgow harbour. It has stood the traffic satisfactorily.

Mr. Joseph Kincaid's iron way has been extensively laid in England. Side-flanged rails are fastened to cast-iron chairs placed at 3 feet apart between centres by means of staples at each side. The staples penetrate into hard-wood plugs let into the chairs. For the Bristol tramways, the rails were of wrought-iron, weighing 43 lbs. per yard; for the Leicester tramways they were of Siemens steel, 47 lbs. per yard. In the more recent development of the system, for the Salford Corporation Tramways, Figs. 101 to 103, the rails, of iron, weigh 50 lbs. per yard; they are 3½ inches wide, and are 2¾ inches deep, with a maximum thickness of 1⅜ inches. The tread, or rolling surface,

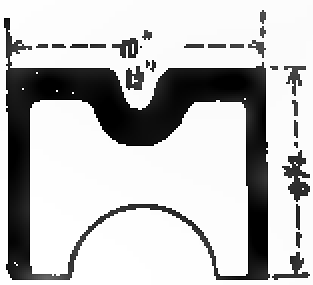


Fig. 100.
Ransome's Tram-rail.

is 1¾ inches wide; it is flat and inclined, so that at the centre of the rail it is ⅛ inch higher than at the side. Car-wheels, consequently, take their bearings on the middle or centre

Fig. 101.—Kincaid's Tramway.

line of the rail. The paving consists of granite sets, 6 inches deep, laid on a bed of sand 2 inches thick.

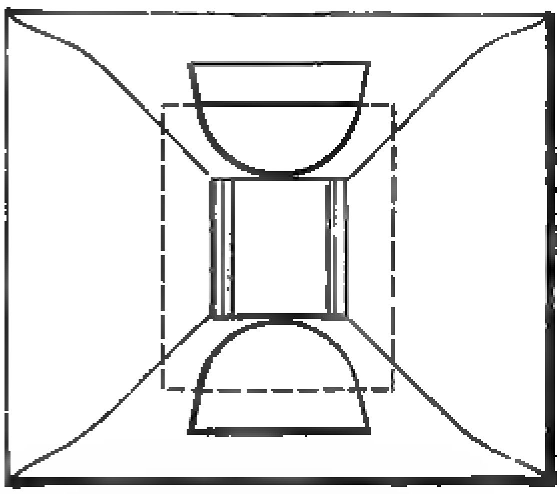


Fig. 102.—Kincaid's Chair.

Barker's way has been laid in Manchester. The peculiar features of this system are the longitudinal cast-iron sleepers, which afford a continuous bearing for the rail and for the adjoining paving sets; and the grooved rail, of which the

lower surface is indented longitudinally, and is formed with a central flange or web, by which it is fastened by cotters to the sleeper. The sleeper is in section like the ordinary bridge-rail in use on railways, but it is of larger dimensions. The general design is illustrated by Fig. 104, adapted for country lines of light traffic. For street lines, in Manchester, the scantlings, as adopted by Mr. J. H. Lynde, are heavier. The sleeper consists of a hollow vertical portion, 9 inches wide, finished with a solid head, formed to fit and to carry the rail; and two horizontal flanges, about 4 or 4½ inches wide, making in all a broad continuous base 12 inches in width. The total height of the combined sleeper and rail is 7⅜ inches. The rails are of steel, 8 inches wide, weighing 40 lbs. per yard.

Fig. 103.—Kincaid's Rail.

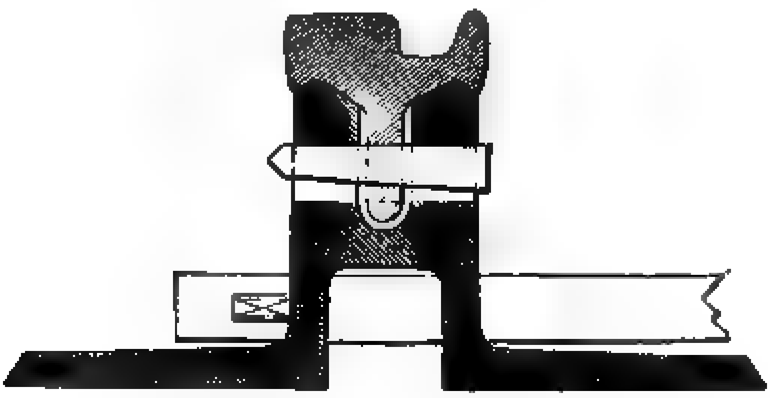


Fig. 104.—Barker's Tramway.

In the Moscow tramways, it appears, the first employment of solid flanged rails of the Vignoles pattern was made. The way, Figs. 105 and 106, was designed by the engineer, Colonel Sytenko, who began by rejecting the grooved rail, and adopted the Vignoles type, laid on transverse sleepers. The

rails are of steel, weighing 36 lb. per yard; they are made to a height of 5 inches, to admit of the juxtaposition of paving-stones of sufficient depth above the sleepers. The paving-stones next the rails at the inner sides are cut to form a

Fig. 105.—Moscow Tramway.

groove for the wheel flanges. The rails are jointed with fishplates and bolts and nuts, as shown in Fig. 106.

Fig. 106.
Tram-rail, Moscow.

Mr. Thomas Floyd, abandoning the three-sided or box rail with the longitudinal timber sleeper, employs a girder rail of the form shown in Fig. 107, having a flange base supported on cross timber sleepers, which have bevelled sides tumbling inwards towards the upper surface. The rail is of steel, weighing 71 lbs. per yard, it is 5½ inches in depth and 5 inches wide at the base. It is rolled complete with the groove. A trench 11½ inches in depth is formed, and of sufficient width, varying with the gauge of the way. The cross sleepers are laid on the bottom at the distances required. On these the rails are spiked down to gauge, and are fished at the joints. The sleepers are then packed up to the proper level by beater picks, and concrete is thrown into the bays between the sleepers and brought up flush with them. The Croydon tramways and the Cambridge tramways have been constructed by Mr. Floyd on this system,

Fig. 107.—Floyd's Tram-rail.

and he contemplates the adoption of the same system for the Woolwich tramways and the Northampton tramways.

In the matter of the paving for the Croydon and the Cambridge tramways, asphaltic pavement has been laid at both places for the sake of freedom from the noise of horses' feet. Mr. Floyd proposed the paving of granite sets, but he was overruled by the local authorities in each instance. The paving is a species of asphaltic macadam; the first layer is made with stone broken to a $2\frac{1}{2}$-inch ring-gauge. The uppermost layer is made with stones broken to a $\frac{3}{4}$-inch or a 1-inch gauge, and is well rolled in. It is finally painted with a mixture of boiling tar and mineral pitch, and strewn with kiln-dried sand. After twelve months' trial of this paving at Croydon, the results were so far satisfactory that its use was continued. Mr. Floyd considers that on roads where vehicular traffic is of a light character, this kind of paving may be used with economy; but that for considerable traffic, granite-set paving is preferable. Taking the cost for asphalte paving at one-third of that of granite paving, in connection with the cost for maintenance, the average annual cost would be equal. The asphaltic paving is liable to ooze upwards during very hot and dry weather, but it keeps the substructure thoroughly dry.

Fig. 108.—Cambridge Tramway.

Mr. Floyd found that the cross-sleepers aid materially in absorbing vibration—a matter of special importance for girder-rails; and that, when such rails are laid on concrete, the greater the sleepers are in width the easier is the motion of the tramcar. The construction of the Cambridge tramway is shown in cross section in Fig. 108. The sleepers are 9 inches wide by $4\frac{1}{2}$ inches deep, and are placed at distances of 4 feet apart between centres.]

CHAPTER XIII.

CANALS.

OF THE GENERAL ARRANGEMENT OF CANALS.

CANALS are artificial channels of water, which have been formed for the purpose of affording the facilities of water conveyance in districts where no natural rivers and streams exist, or where those which may have existed have, from a variety of causes, been ill-adapted for navigable purposes. And, in fact, canals possess (generally speaking) so many advantages over rivers, that they have frequently been constructed, at considerable cost, in situations where navigable rivers were already existing. In many rivers the existence of currents and shoals renders the navigation difficult and uncertain, and in times of floods and freshets, it has frequently to be entirely suspended. It may also be remarked that rivers seldom flow in a very direct course, but more frequently pursue a winding path, depending upon the form of the valleys through which they have to thread their way: in such situations as these, the superiority of canals is sufficiently obvious.

In laying down and arranging the general line of a canal, many points have to be considered in addition to those which have been generally mentioned, as applying to them in common with roads and railways, at the commencement of this chapter. One of the most desirable points to be attained is a perfectly level surface throughout its whole extent. It is, however, very seldom that the country is so favourable

as to allow this to be effected. In most cases it becomes necessary occasionally to alter the level of the surface of the canal, the water being retained at the higher level by gates so placed that the pressure of the water against them keeps them closed. It is, however, impossible to prevent a small amount of leakage at the gates, and therefore it becomes necessary to have the means of supplying the upper portion of the canal with water, to compensate for that which thus escapes, as well as that which is necessary (as we shall presently explain) to pass vessels from the higher to the lower level. In addition to these two causes of loss, a further waste is occasioned by the evaporation from its surface, and the absorption of the water by the ground through which it flows. It is, therefore, an object of considerable importance in the arrangement of a canal, to obtain some natural *feeder* (as it is termed) for the supply of the water thus lost, and which object is usually attained by diverting some of the smaller natural rivers or streams, and leading as much of their waters as may be required to supply the highest (technically called the *summit*) level of the canal, for that being properly supplied, the lower levels will be fed by the water which escapes from the upper. Before forming a canal, the strata through which it will pass should be carefully examined, more especially with reference to its powers of retaining water, that is, of not absorbing it. Many soils, such as clean sand, or gravel, would carry off the water so rapidly as soon to drain the canal, and therefore such strata should, if possible, be avoided. Where, however, it is impossible to do so, the canal may be made water-tight by lining its sides and bottom with *puddled* clay, which consists of good clay, thoroughly well beaten up with water, or *tempered*, and then mixed with a certain proportion of gravel, sand, or chalk. Pure clay by itself would not answer, because if at any time the water in the canal sunk below its ordinary level, the upper part of the puddle, becoming dry,

would crack ; and when the water again rose it would escape through these cracks, which by its action would be gradually enlarged, until the puddle was rendered useless.

The form of section of a canal, that is, its width and depth, is another point requiring to be carefully considered. This must depend upon the size of the vessels which are to be conveyed upon it, and upon the amount of the traffic to be expected. The sides of canals are usually formed with slopes, of about two to one, and, in some cases, the upper parts, near the water's edge, and which are most exposed to the ripple produced by the passage of vessels, are protected by rough stone paving.

The following table exhibits the length and dimensions of the transverse section of a few of the English and American canals :—

NAME OF CANAL.	Date of construction.	Length in Miles.	Breadth. Top.	Breadth. Bottom.	Depth.	ENGINEER.
ENGLISH.			Feet.	Feet	Ft. In.	
Sankey Canal	1755	12	48	—	5 7	John Eyes.
Leeds and Liverpool . .	1770	108¾	42	27	5 0	Brindley.
Basingstoke	1778	37	38	—	5 6	—
Thames and Severn . .	1783	30	42	30	5 0	R. Whitworth.
Gloucester and Berkeley	1793	16½	70	—	18 0	Telford.
Grand Junction	1793	90	43	—	5 0	Jessop.
Kennet and Avon . . .	1794	57	44	24	5 0	Rennie.
Aberdeenshire	1796	18¼	23	—	3 6	Captain Taylor.
Thames and Medway . .	1800	8½	50	28	7 0	—
Caledonian	1803	23	40	—	20 0	Telford.
Rye, or Royal Military .	1807	30	72	36	9 0	Royal Engineers
AMERICAN.						
Champlain	—	11	40	28	4 0	—
Schuylkill Navigation .	—	58	36	22	3 6	—
Morris	—	101¾	32	20	4 0	—
Pennsylvania	—	276¼	40	28	4 0	—
Erie	—	363	40	28	4 0	—

Of Locks and their Substitutes.

We have already mentioned that, in cases where it is necessary to alter the level of the surface of a canal, the water is retained at the higher level by means of gates ; and

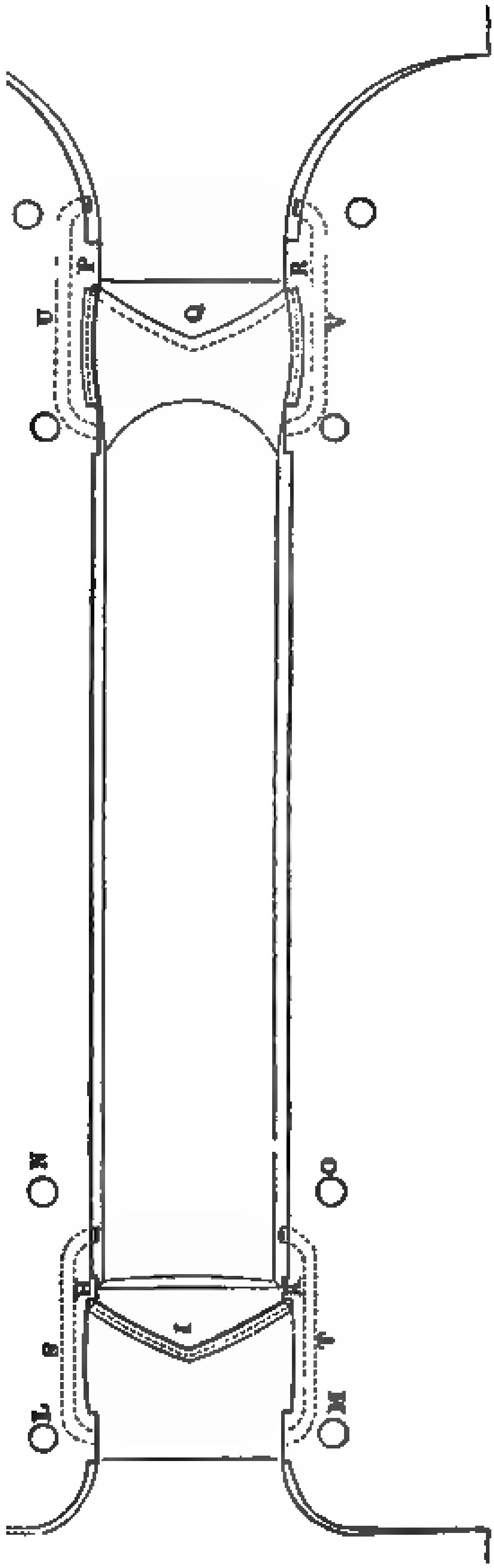


Fig. 109. Canal Lock. Fig. 110.

we have now to explain more in detail the manner in which they are constructed, as well as the means adopted for passing vessels up or down from one level to the other.

The most frequently employed contrivance for this purpose is the common *lock*, of which Fig. 109 is a longitudinal section; Fig. 110 a plan; Fig. 111 a transverse section through the centre of the lock; and Fig. 112 a transverse section of the canal below the lock, showing its lower entrance. The upper and lower portions of the canal are connected by the passage A B C, termed the lock chamber, the form of which will be seen from Fig. 110; its sides and bottom (the latter termed the *invert*, or *floor*) are usually lined with brick or stone. The lock chamber is much less in width than the canal, being made only a little wider than the vessels intended to pass through it. It will be observed, by reference to Fig. 109, that the floor of the upper end of the lock chamber, from D to E, is on the same level as the upper portion of the canal; and the remainder, from F to G, is level with the bottom of the lower canal. The gates, by means of which the water is retained at the upper level, are shown at A E, Fig. 109, and in the section, Fig. 112; they are slightly curved, as shown in the plan, Fig. 110. When opened, they turn upon their ends, H and K, as centres; and they are of such a breadth that, when shut, they meet at an angle at I, in which position each gate derives support from the other; and the pressure of the water against them only tends to keep them the more closely shut, and, consequently, to diminish the space through which it might otherwise have escaped.

The gates are opened by means of capstans, L and M, the chains being attached to the gates under the water, and passing through tunnels in the sides of the lock. They are closed in a similar manner, by two other capstans, N and O, the gate H I being shut by means of the capstan O, and K I by means of N.

Another pair of gates, precisely similar, are placed at the

lower end of the lock, C G; they are carried up to the same level as the upper gates, and are therefore as much higher than these as the upper canal is above the lower, as is shown at A and C, Fig. 109, and in the two sections, Figs. 111 and 112.

We will now proceed to explain the mode in which the lock is used; and we will first suppose the case of a boat requiring to be raised from the lower to the upper level of the canal. The lower gates, at C, are first opened, as shown in Figs. 109 and 110, and the boat is floated into the lock chamber (the length of which should be a few feet more than that of the longest boat passing along the canal); they are then shut, and brought into the position shown by the

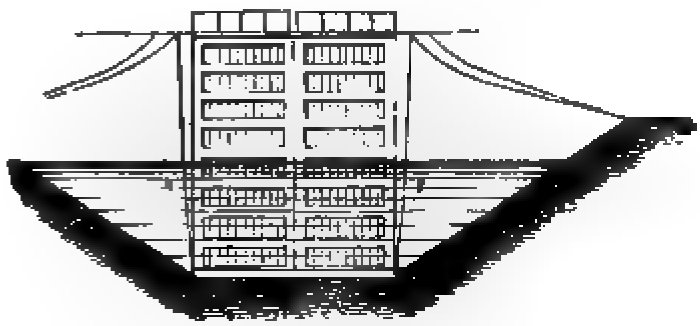


Fig. 111. Canal Lock. Fig. 112.

dotted lines, P, Q, R, in Fig. 110, which having been done, some of the water from the upper canal is let into the lock chamber, through chambers shown at S and T, in the sides of the upper part of the lock, and which can be opened and closed at pleasure, by sluices worked by machinery. The water being prevented from flowing out, in consequence of the lower gates being shut, quickly rises to the same height in the lock chamber as in the upper canal, the boat rising with it. As soon as such is the case, the upper gates at A are opened, and the boat is floated out of the lock into the upper canal. The reverse operation of lowering a boat from the upper to the lower level is performed in a similar manner; the boat is floated into the lock chamber, the gates

at A being opened, and those at C closed; the former are then shut, and the water in the lock chamber is allowed to run out by channels, U V, formed at the lower end of the lock, similar to those already described at the upper, until level with the surface of the lower canal, when the gates at C are opened, and the boat passes out of the lock.

The quantity of water let out of the upper canal in the passage of a boat depends upon the direction in which the boat is moving, and whether it finds the lock filled or empty. The following Table shows all the cases which can occur:—

	Finding the Lock,	Lets out of the Upper Canal,	And leaves the Lock,
Boat descending .	Full . .	None . . .	Empty.
	Empty .	1 Lockfull .	
Boat ascending .	Full . .	1 Lockfull .	Full.
	Empty .	1 Lockfull .	

It is therefore evident, that a series of boats following each other in the same direction, either up or down, will require one lockfull of water for every boat that passes; but if the boats pass alternately up and down, only one lockfull will be required between each pair, since every ascending boat requires a lockfull, and leaves the lock full; and every descending boat finding the lock full, does not require any water from the upper canal.

When the ground rises or falls so rapidly as to require several locks in a short distance, it is not unusual to form what is called a *chain of locks*, or to make a succession of lock chambers immediately contiguous to each other, the lower gates of the chamber forming the upper gates of the next below it, as shown in Fig. 113. The advantage of this arrangement is a considerable saving in the cost of constructing the locks, arising from the circumstance that only one

more than half the number of gates, with all the machinery for opening and closing them, is required.

In some situations, where the supply of water for lockage is small, a system has been adopted by which the quantity required for this purpose is much lessened. This system consists in forming one or more excavations or ponds by the side of the lock chamber, with which they are connected by culverts, having sluices, or valves. The level of these ponds is so arranged that when the lock is full, and it is desired to let off the water, so as to lower its surface to the level of the lower canal, instead of allowing the whole of the water to run into the canal, a portion of it is run into the pond, and

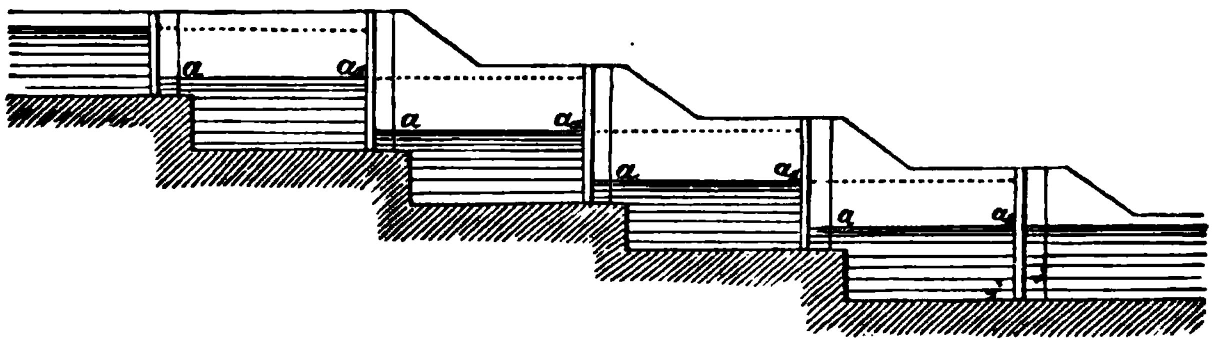


Fig. 113.—Canal Locks.

there kept until it is again desired to fill the lock chamber, when, instead of taking the whole of the water required for that purpose from the upper canal, that from the pond is first allowed to run into the lock, and the remainder only taken from the upper canal.

Canal Aqueducts.

In carrying canals across short and deep valleys, in order to avoid a succession of locks which would be required if the surface of the canal were made to conform to that of the valley, it is usual to carry them across at a higher level, through a water-tight channel formed and supported upon arches. Such structures are termed *aqueducts*, and in their construction have afforded some fine opportunities for the display of engineering skill.

Figure 114 is an elevation of a portion of one of the most celebrated aqueducts, that of Pont-y-Cysyllte, constructed by Telford, for the purpose of carrying the Ellesmere and Chester Canal across the valley of the Dee. It is upwards of 1000 feet in length, consisting of nineteen arches of equal

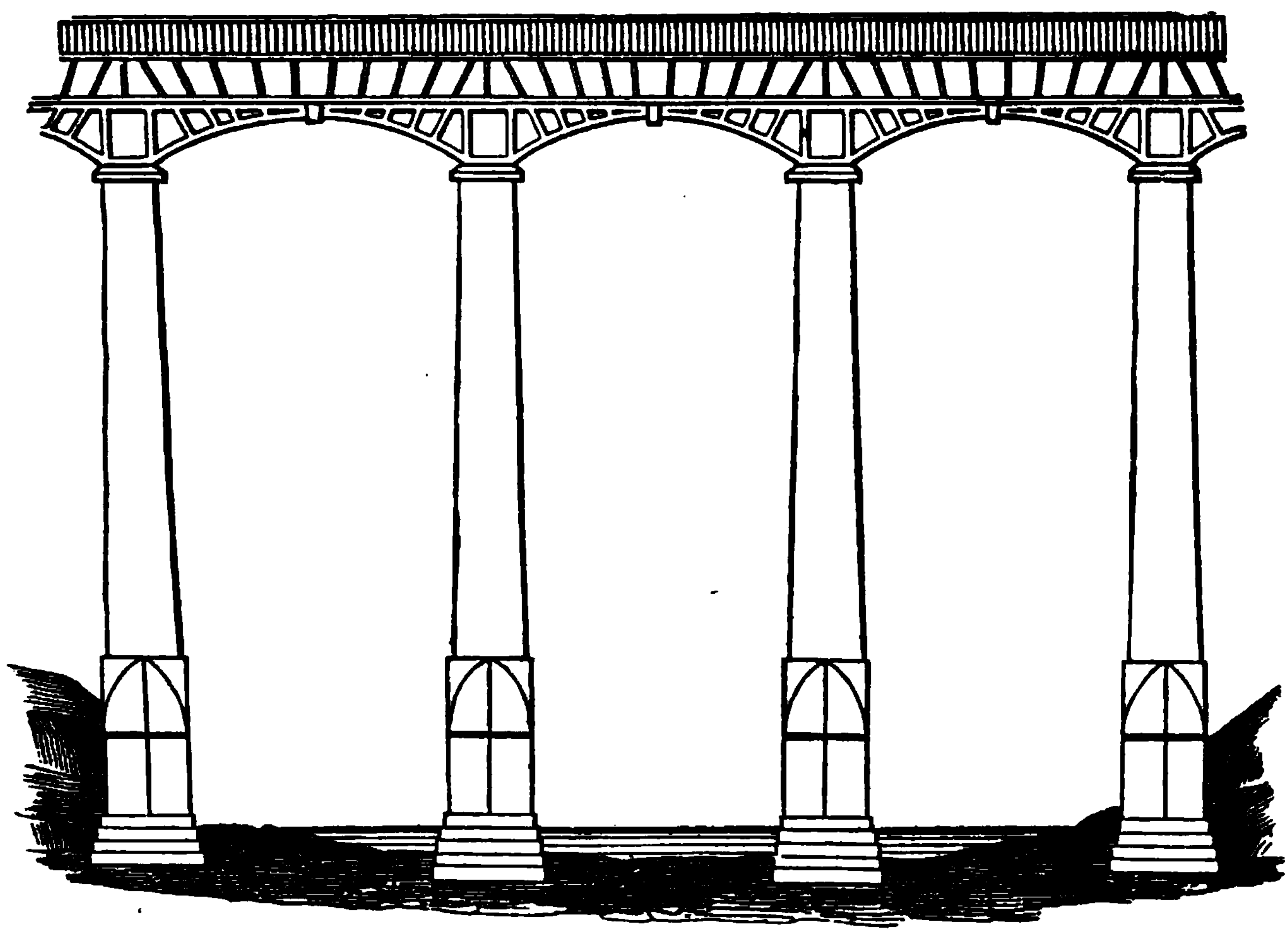


Fig. 114.—Canal Aqueduct.

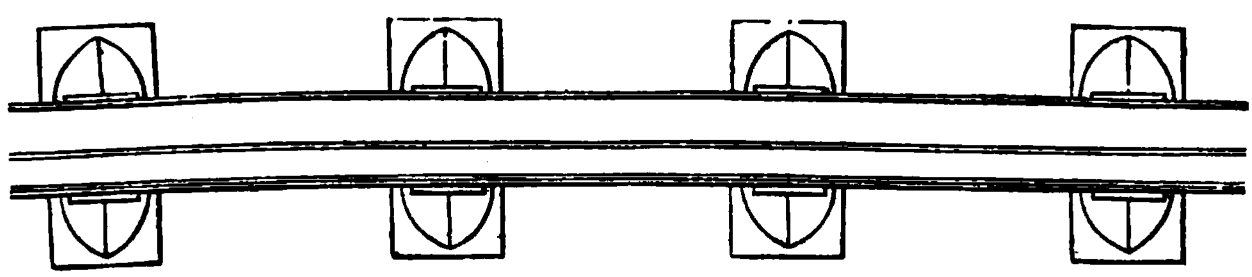


Fig. 115.

span, but varying in their height above the ground. The three shown in elevation in Fig. 114, and in plan in Fig. 115, are the highest, being those which cross the River Dee itself; the surface of the canal is 127 feet above the usual level of the water in the river. The aqueduct itself is a cast-iron trough (shown in section in Fig. 116), formed of plates with flanges

securely bolted together. This trough is supported upon cast-iron arches, each composed of four ribs, supported upon piers of masonry. The towing path overhangs the water, being supported at intervals on timber pillars, as shown in Fig. 116.

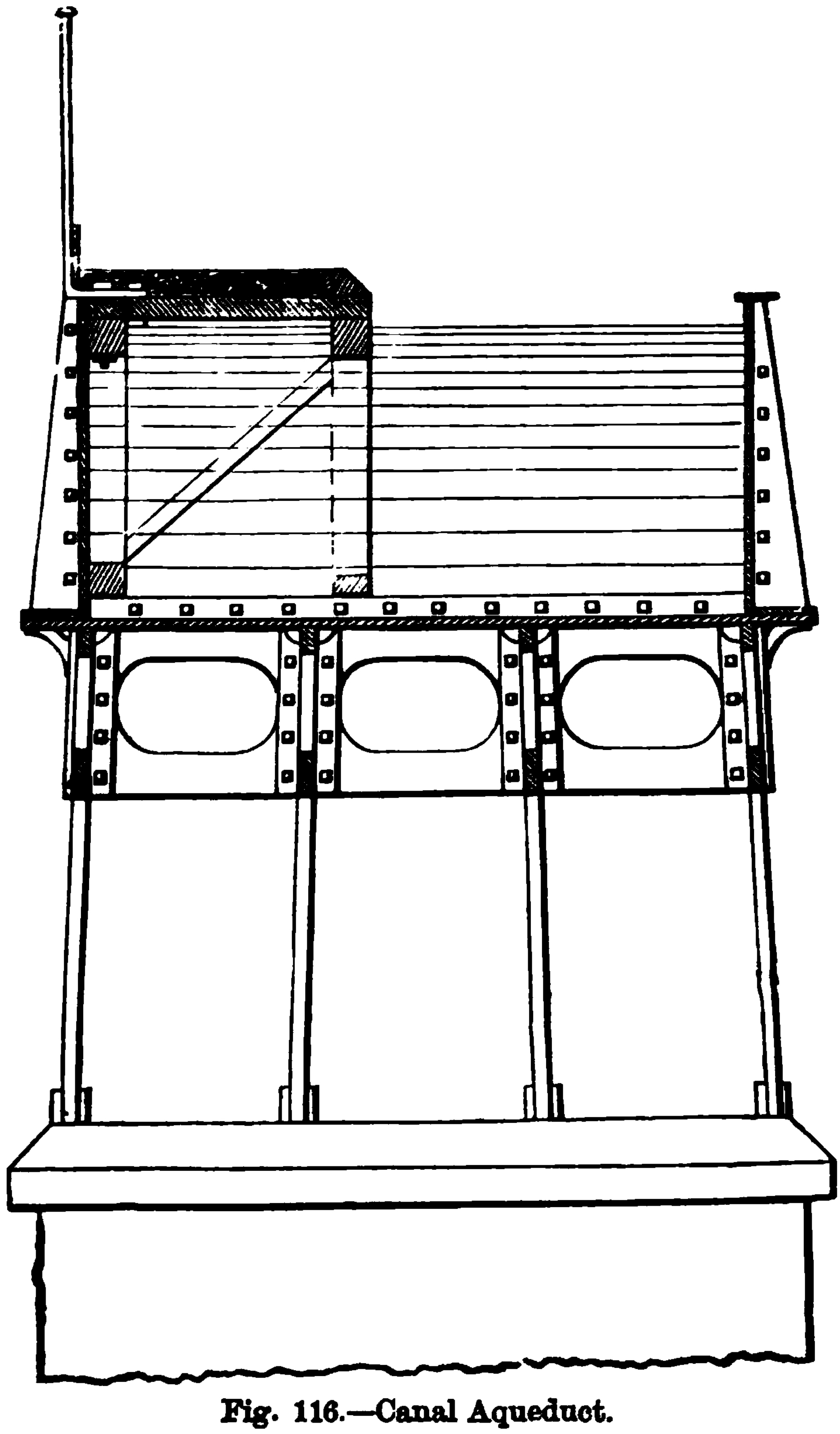


Fig. 116.—Canal Aqueduct.

Fig. 117 is a transverse section of the Chirk Aqueduct, carrying the same canal across the valley of the Creiroig, at a height of 70 feet above the level of the river beneath. It consists of ten arches of equal span, constructed of masonry;

in this case only the bottom or floor of the canal is of iron; the sides, which are 5 feet 6 inches in thickness, being built of ashlar masonry backed with brickwork in cement.

[Canals are classified as barge canals and ship canals; the former laid out for local or inland traffic, the latter for through traffic from sea to sea by ships. Barge canals, notwithstanding the competition of railways, appear to hold their place in the system of inland transport of goods and minerals, particularly in the United States and in Canada, where canal extensions are made. Even in railway-ridden England the Birmingham, Grand Junction, and other canals appear to carry on as brisk a trade as ever. But it is not likely that they will be extended in competition with railways; for they are neither quick nor altogether certain in the matter of water supply, particularly in dry seasons; nor in severe winter weather, when the traffic is liable to be interrupted by ice. Such objections do not apply in the same degree to ship canals, of which the low-level canals receive their supply from the sea, and which cannot, at least in temperate climates, be frozen over; whilst ship canals generally command a monopoly of traffic in affording short and sheltered passages for sea-borne vessels.

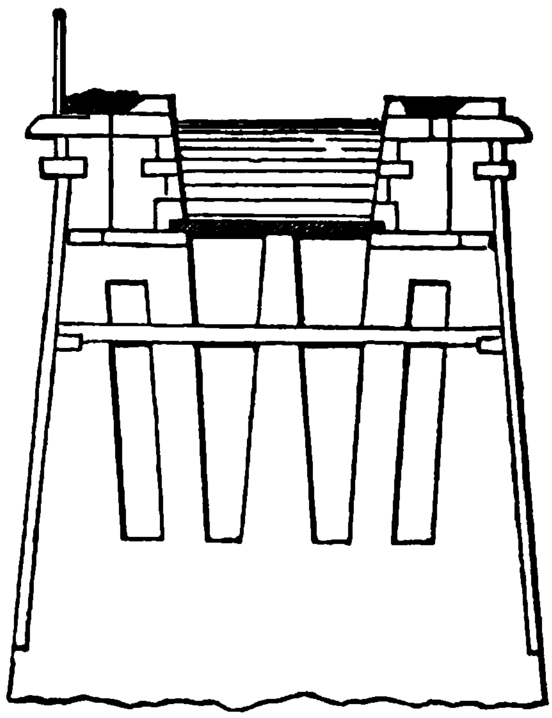


Fig. 117.—Canal Aqueduct.

Barge Canals.

According to the section generally adopted for barge canals, they are constructed with a width of from 24 feet to 40 feet, and are from 4 to 5 feet in depth. When formed

in a retentive soil, they are made as shown in section in Fig. 118, having a towing-path at one side; but when the soil is porous, clay puddle is introduced, as shown in Fig. 119. That there should be no material augmentation of the resistance of a boat, beyond the normal resistance in open water, the breadth at the bottom should be at least twice the greatest breadth of the boat, the depth should be at least 18 inches more than the draught, and the sectional area of waterway should be at least six times the greatest midship section of the boat.

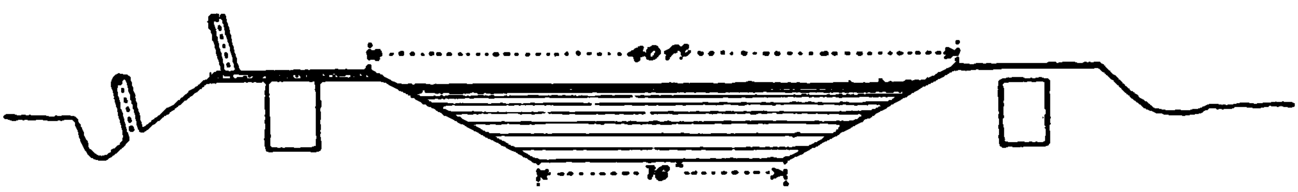


Fig. 118.—Canal.

Locks on barge canals in England have a width of 8 feet, and they are from 70 to 80 feet in length, with a lift ordinarily of 8 feet.

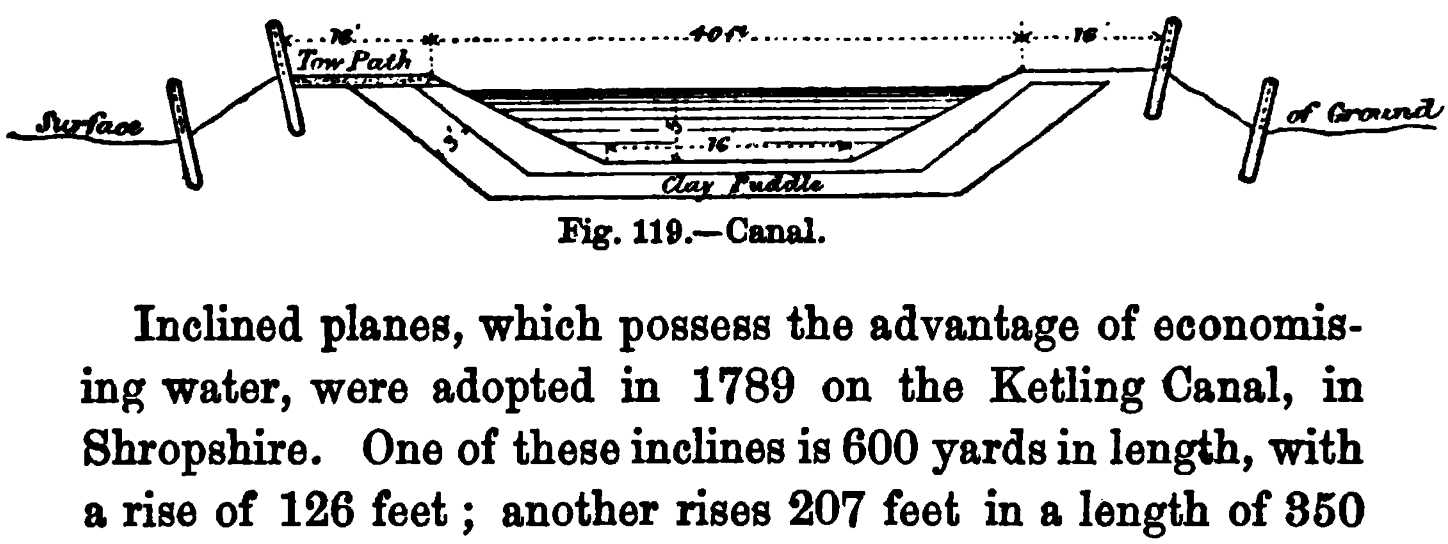


Fig. 119.—Canal.

Inclined planes, which possess the advantage of economising water, were adopted in 1789 on the Ketling Canal, in Shropshire. One of these inclines is 600 yards in length, with a rise of 126 feet; another rises 207 feet in a length of 350 yards. The boats, which carry about 5 tons each, are drawn by machinery on a railway laid on the incline.

Mr. Douglas, of New York, constructed the Morris Canal, in the United States, between the rivers Hudson and Delaware, with 23 inclined planes, having gradients of about 1 in 10, with lifts averaging 58 feet. The boats, with their load, weighed 50 tons, and after having been grounded on a carriage, they were raised by water-power up the inclines with

ease and expedition. The length of the canal is 101 miles, and the total rise and fall is 1,557 feet, of which 223 feet are effected by means of locks, and the remaining 1,334 feet by inclined planes. Slightly built boats 80 feet long are liable to injury by straining while resting on the cradle; but this objection has, to some extent at least, been overcome by Mr. Leslie and Mr. Bateman on the Monkland Canal, where the boats are not wholly grounded on the carriage, but are floated and transported in a carriage of boiler-plate containing 2 feet of water. This inclined plane is 96 feet in height to a gradient of 1 in 10, and is worked by two 25 horse-power steam-engines. The maximum weight raised is 80 tons, and the transit is effected in ten minutes. The average total number of boats passed over the incline is about 7,500 per year.

The most recently constructed lift for canal boats is that designed by Mr. E. L. Williams, jun., for establishing a means of communication between the river Weaver and the Trent and Mersey Canal at Anderton. The canal, which for some miles runs parallel and close to the river, is on the top of a bank, whilst the river runs at the bottom, at a level of 50 feet 4 inches below that of the canal. There is an island in the Anderton basin of the Weaver, which was fixed upon as the site of the lift. There is also a basin of the canal, from which the water of the canal is carried in a wrought-iron aqueduct at the level, across an arm of the river, to the end of the lift-pit on the island, where the boats are lifted and lowered between the end of the aqueduct and a cutting from the main river into the island. The aqueduct is of wrought iron, 34 feet 4 inches wide, 8½ feet deep, divided longitudinally into two channels by a central web, and carrying 5¼ feet of water. Each end of the aqueduct is fitted with wrought-iron balanced lifting-gates for controlling the ingress and egress of barges. The lift is double, and the barges are raised or lowered while floating in a box or trough full of

water: so arranged that one trough containing barges, in coming down to the river, assists in lifting barges in the other trough up to the canal. The troughs are 75 feet long and 15½ feet wide, holding 5 feet of water—long enough to hold the largest barges that can be used on the canal, and wide enough to hold one of the largest barges carrying from 80 to 100 tons, or two of the small ordinary barges carrying from 30 to 40 tons. Each trough is attached to the head of a vertical cast-iron hydraulic ram, 3 feet in diameter, by means of which it is raised and lowered. The gross load on one ram, comprising the weight of one trough, with water and barges, amounts to 240 tons, equivalent to a pressure of 4¾ cwt. per square inch of the area of the ram. The presses are below the bottom of the lift-pit, within cast-iron cylinders sunk to a depth of 70 feet. An accumulator assists in working the lift, having a 21-inch ram, with a stroke of 13½ feet, and a capacity equal to that of one of the main rams for a stroke of 4½ feet. Besides being worked as a double lift, each trough can be lifted separately by the engine and the accumulator—an operation requiring half an hour. But when the two lifts work in conjunction, the operation of raising and lowering simultaneously occupies from 2½ to 3 minutes. The motive power is of two kinds. First, eleven-twelfths of the entire lift is performed by using a layer of water, 6 inches deep, from the upper trough; second, the remaining twelfth is supplied by engine-power, by which water is continually pumped into the accumulator. By means of these combinations, an economy of time, water, and attendance is effected. But it so happens that the descending loads are so much greater than the ascending loads, that in practice there is no loss of water from the canal. The lift was opened for regular duty in July, 1875. The system possesses two obviously good features. The barges are maintained in a state of floatation, and it is impossible for barge owners, however rotten the barges may be, to say that

they had been damaged. Again, chains are entirely dispensed with, as the lift is performed entirely by the direct action of the press. The cost of the ironwork and machinery was £29,463; foundations, basins, and approaches, £18,965; together, £48,428. The contract for the ironwork was let in 1872, when maximum prices prevailed. In full operation, the working expenses amount to £15 per week; adding 10 per cent. of the prime cost, £93 per week, makes a total of £108 per week. The lift is capable of transferring 16 barges per hour—8 up and 8 down—equivalent to a total of 960 barges transferred per week. The laden barges average about 25 tons burden each, making 12,000 tons per week, giving as the average working cost 2·16d. per ton.

The rapidity of performance of the lift, by which in eight minutes two barges can be transferred from the river to the canal, and two others from the canal to the river, is illustrated by comparison with the operations at a flight of locks on the canal at Runcorn, where it requires from 1¼ to 1½ hours for a barge to pass through the locks.*

A sufficient number of waste-weirs, for the discharge of surplus water accumulating during floods, are required on canals. Wherever the canal crosses a stream, and at other points where the canal is liable to influx, waste-weirs should be provided, with courses for discharge of the water into the nearest streams. Waste-weirs are placed at the top water levels, so that when a flood occurs the water overflows directly, and the banks of the canal are relieved. If suitable exits be not provided, the banks may be breached, the tow-path may be flooded, adjoining lands may be damaged, and the traffic may be arrested.

Stop-gates are necessary at short intervals of a few miles, for the purpose of dividing the canal into isolated reaches, in

* See a paper on "Hydraulic Canal Lift at Anderton, on the river Weaver." By S. Duer. *Proceedings of the Institution of Civil Engineers*, vol. xlv., p. 107.

order that, in the event of a breach, the gates may be closed on the defective portion and the water run off from that part, for the execution of repairs, whether special or general. Stop-gates may be constructed simply of thick planks, which are slipped into grooves formed at those narrow parts of the canal which occur under wood bridges, or at contractions made at intermediate points to receive them. Mr. D. Stevenson instances an example of the value of stop-gates in obviating serious accidents. The water during a heavy flood flowed over the towing-path of the Union Canal, connecting Edinburgh and Glasgow, near the end of an aqueduct which adjoined a high embankment. The uncontrolled

Fig. 120.—Canal.

current carried away the embankment and the soil on which it rested, to a depth of 80 feet below the top water-level. The stop-gates were promptly applied, and the overflow and the consequent damage were confined to a short reach of a few miles.

For the purpose of draining off the water to admit of repairs after the stop-gates have been closed, "off-lets," or discharge-pipes, are placed at the bottom of the canal, fitted with valves, which can be opened or closed when required. Off-lets are generally found at aqueducts or bridges crossing rivers, where the water may be run off into the stream.

The tow-path should be made with a gentle inclination downwards from the canal towards the inner side, for the

purpose of drainage, and also to enable the horses better to resist the oblique pull of the boats. The drainage of the tow-path should be carried to a sky-drain, as in Fig. 120, and at intervals passed below the path to the canal.

The protection of the banks at the water-line is a matter of importance, as the washing, or waves, created by passing boats extends 9 inches or 12 inches above and below the still water-line. "Pitching" with stones or facing with brushwood is applied, as indicated in the figure. The latter system forms an economical and effectual protection.

Ship Canals.

The Caledonian Canal was constructed through the "Great Caledonian Glen," to supersede the coasting voyage by the north of Scotland, through the stormy Pentland Firth. The district embraces a chain of fresh-water lakes, which are connected by reaches of canal. It was constructed 20 feet deep, 120 feet wide at the top, and 50 feet at the bottom; but the working depth has recently been given by Mr. D. Stevenson as 18 feet. The canal is capable of transporting vessels 160 feet in length, 38 feet beam, with a draught of 17 feet. The total length of the passage is 60½ miles, of which 23 miles consist of artificial canal, and the remainder of lake navigation. The summit level at Laggan is 102 feet above the level of neap tides, and is reached by means of 26 locks—13 locks on either side—having a lift of 8 feet. The locks are 170 feet long and 40 feet wide. The cost of the canal amounted to a million sterling. The canal was opened in 1823.

The Languedoc Canal, by a short passage of 148 miles, saves a sea voyage of 2,000 miles by the Straits of Gibraltar. By the Forth and Clyde Canal, making 35 miles of inland navigation, sea-borne vessels may be passed across Scotland. The Crinan Canal substitutes a short inland route across the Mull of Kantyre for a sea voyage of 70 miles round.

The Amsterdam Canal, 15¼ miles long, was constructed between Amsterdam and the North Sea, through the Wyker Meer. The canal passes from the North Sea by a deep cutting through a broad belt of sand hills, which protect the north coast of Holland from the inroads of the sea, and then enters the Wyker Meer and other tracts of tide-covered land, whence it reaches Amsterdam. The material excavated from the cuttings was deposited so as to form two banks 443 feet apart, through the lakes on each side of the main channel, thereby leading to the reclamation of 12,000 acres of land. To provide for the drainage of the land, the Canal Company are bound to maintain the surface of the water in the canal 1 foot 6 inches below the average high-water level. For this purpose large pumps, worked by engines of 180 horse-power, and capable of discharging 2,700 tons of water per minute, are used. A sufficient barrier is provided against the sea at each end: the sea level at high water being occasionally several feet above the level of the canal. The entrance-locks at each end of the canal are for the purpose of locking downwards, not upwards. They have three passages for vessels, of which the central passage is 60 feet wide and 390 feet long, and is furnished with two pairs of gates at each end, pointing in opposite directions, and one pair at the centre. The gates pointing seawards are of cast-iron, the others, pointing inwards, are of wood.

There is but one ship-canal—the Suez Canal—free from locks, and communicating freely with the sea at each end, connecting the Mediterranean Sea with the Red Sea. It is a short cut, 88 miles long, by means of which the communication between Western Europe and India has been reduced in length from 11,379 miles, by the Cape of Good Hope, to 7,628 miles. Of the whole length, 88 miles, 66 miles are actual canal formed by cuttings, 14 miles have been made by dredging through the lakes, and 8 miles did not require any works, as the natural depth was equal to that of the canal.

The channel was excavated partly by dredging, and partly by hand labour, the stuff being deposited on each side to form banks. The canal, shown in section, Fig. 121, is 72 feet wide, bounded by slopes of 2½ to 1, then a berme of 50 feet on each side, and slopes of 3 to 1 and 5 to 1, so as to form a flat beach, on which it was anticipated that the wave from passing vessels could expend itself without injury to the banks. The canal is capable of receiving vessels 400 feet long, of 50 feet beam, and 25 feet draught. The water, which was at one time noted for its extreme saltness, is gradually losing this characteristic, no doubt because the salt deposits in the Bitter Lakes are gradually melting away. The shores of the canal are [1879] in course of being faced with stones, to preserve the banks from the action of waves. The canal is affected by the sandstorms which at certain times of the year prevail. As the steamer jogs quietly along, at the regulation speed of 5·80 knots an hour, all at once one perceives whirlwind after whirlwind, in quick succession, sweeping over the desert, their presence indicated by a column of sand rising far into the air and darkening the sky. Occasionally one of these sandstorms crosses the canal and discharges clouds of dust into it, coating the passing vessel with sand more than an inch thick. By the work of dredging the canal is nevertheless kept clear. The canal was opened in the end of 1869. The

Fig. 121.—Suez Canal.

deep channel through the lake is marked by iron beacons on each side, 250 feet apart. There are passing places at intervals of 5 or 6 miles, to admit of large vessels mooring for the night, or to bring up in order to allow others to pass. At each passing place a telegraph station is erected, with an officer to regulate the movements of vessels. In the southern portion of the canal, between Suez and the Great Bitter Lake, the tidal influence from the Red Sea is felt, as there is a regular flow and ebb. The rise at spring tides is between 5 and 6 feet at Suez, and about 2 feet about six miles inland; at the Small Bitter Lake, a few inches only. It is stated that, in the execution of the works of the canal, there have been excavated about 80,000,000 cubic yards of material. At one time nearly 30,000 labourers were at work. A supply of fresh water was brought from Cairo for their use, by a fresh-water canal, *viâ* Zagazig. The terminal harbour in the Mediterranean Sea, at Port Said, is formed by two breakwaters constructed of concrete blocks, and enclosing an area of about 450 acres. The entrance at Suez is also protected by a breakwater. In connection with the harbour there are two large basins and a dry dock. The total cost of the works of the Suez Canal amounted to about £20,000,000 sterling.]

CHAPTER XIV.

RIVERS.

Rivers present, in the whole of their course, from the point where they rise to that at which they fall into the sea or into some other river, the following circumstances:—their width increases as they advance, and their longitudinal section, excepting in some extraordinary cases, consists of concave curves, both at the bottom of the beds and at the surface line, although these curves are not necessarily concentric or parallel to one another. The courses of all rivers are so devious that it is an invariable rule that their length, measured upon their longitudinal profile, is greater than the rectilinear distance between their extremities. If the river fall into a sea, or another river, whose levels are exposed to variations, whether periodic or not, the transverse and longitudinal sections of the one thus falling in are exposed to variations beyond the influence of their own waters. Should the variations of the receiving channels be subject to tidal action, the subsidiary rivers will follow the usual laws; the neaps and the springs, the ebbs and the floods, will act upon them in an analogous manner, but in a different degree, to what they do on the sea.

The manner in which rivers fill up, or raise, their beds, is a subject involved in some obscurity, or at least it depends upon causes which are often purely local. In many rivers the tendency of the water is rather to lower the bed, especially when it runs upon hard rocks, than to deposit the

detritus brought down from the upper districts; and this tendency to deepen the beds is principally confined to the upper and more rapid portions of the course. The detritus in these portions is deposited in the various small branches, or bays, or, in fact, in any positions where a sudden change takes place in the rate of flow; and, when this law is skilfully applied by the engineer, it may be made to co-operate very efficaciously in the improvement of the course of the stream. But in the lower portions of the river, where the descending velocity of the water is destroyed by the meeting with the sea, the sand and mud are deposited gradually all over the surface of the bed, giving rise to the deltas which are so characteristic of the mouths of rivers, particularly in tideless seas, such as the Mediterranean and the Gulf of Mexico.

According to the natural laws of gravity the velocity of the waters in a river would continually increase, agreeably to the rates of inclination of its bed, did not the friction upon the sides and the bed increase at the same time with the velocity, and in a much greater proportion. The friction also modifies the rate of flow of the several separate portions of the transverse section, causing it to be greater in proportion to the depth or volume over any particular part of the contour. There is, in almost all rivers, a zone where the depth is greater than in the other parts, and where, consequently, the velocity is greatest; this zone is called the "thalweg" by foreign engineers, and forms usually the navigable channel. Beyond it there are frequently other zones of still water, and in some cases these are characterized by currents flowing in an opposite direction to that of the main stream. In the thalweg itself, also, the velocity is not the same at the bottom that it is at the surface, where in rivers of ordinary depths it is at the maximum. It is usual to consider the mean velocity to be about four-fifths of that of the maximum.

The following table is extracted from the "Cours de Construction," by Sganzin; it shows the velocity of some of the most important rivers, principally in western Europe.

			ft.	in.
Mean velocity of the Seine, below Paris	..	per second	2	3
" " Thames at London, flood tide		"	3	0
Velocity of the Tiber at Rome, low water	..	"	3	4
" Danube at Ebersdorf "	..	"	3	6
" Loire "	..	"	4	4
" Rhone at Arles "	..	"	4	10½
" " Beaucaire "	..	"	8	6
" Durance below Sisteron "	..	"	8	6
" Maragnon, S. America "	..	"	13	0
" Rhine varies from 3 ft. 2 in. to about		"	14	0
" a torrent produced by the melting of snow by the sudden action of a volcano	.. ..	per second	25	7

From the circumstances connected with the origin and subsequent flow of rivers, it follows that their volumes are exposed to considerable variations. Thus, the melting of a fall of snow, or a sudden storm, may cause the waters to rise in a very anomalous manner, producing in those parts of the course which are beyond the influence of tidal action serious modifications in the velocity and depth of the water, as well as in the cross section. It becomes important, therefore, before commencing any works for the improvement of a river, to ascertain the precise range of its variations of volume, and the numerous causes which may affect, not only the district drained by the principal stream, but also those of its affluents. Indeed, when rivers are of great length it frequently happens that the floods of the various subsidiary hydrographical basins occur at very distant epochs, and introduce numerous causes of irregularity in the flow. As, for instance, in the case of the Mississippi, the freshets from the upper valleys of the Mississippi and Missouri come down at different periods from those of the Ohio and Tennessee valleys, and, generally speaking, at a later period of the year. It is observed that the floods of the Ohio, under these cir-

cumstances, cause the waters of the Mississippi to be, as it were, penned back for a considerable distance; and equally the floods of the Mississippi occasionally pen back the waters of the Ohio for many leagues. The same remark will apply to most great rivers; but in our own country the extent of the hydrographical basins is not sufficiently great to allow of much irregularity of this description; and we may consider with tolerable safety that our rivers, above the influence of the tides, are at the lowest in the months of June, July, August, and September, and that the floods occur in the months of December, January, February, and March.

Under ordinary circumstances we find that the banks of a river resist less than the bottom, and that the width proportionally is greater than the depth. The tendency of the constituent particles of the banks to fall down by the effect of gravity adds to this excess of the one dimension over the other; and as the larger and more solid materials thus carried down from the sides remain at the bottom, they also serve to augment its stability by their greater resistance. In long level plains the velocity of the stream necessarily diminishes, and any accidental obstacle acquires increased power to deflect it from its natural course, which would be upon the line of greatest longitudinal fall. Should the bank be of a harder and more resisting nature on one side than the other, or should any natural or artificial projection exist, the stream will turn towards the other side; and its bed may thus become sinuous, and present such an increase of length as materially to retard the flow of its waters. In winter, it is also to be observed that, if the upper surface be frozen over, the whole of the abrasive action of the stream is exercised upon the bottom, which it will deepen so long as the water thus flows, as it were, in a pipe.

It follows from what has been said above, in this and the preceding chapter, that the works required for the improvement of the channel of a river may be directed either to

regularise its flow in such a manner as to retain a sufficient depth of water for the purposes of navigation or of adaptation to manufacturing or irrigation uses; or simply to defend the surrounding country from the ravages of inundations, whether they be caused by floods from the upper districts or by the tides.

The first inquiries to be made in either case must be directed to ascertain all the variable conditions of the flow and volume of the river, the nature of its bed, and both its plan and section. As far as regards the adaptation of any stream to manufacturing and irrigation uses, the principal point to be decided will always be the height to which the water may be penned back, because evidently upon this, to a great extent, will depend the power it can produce and the surface it can irrigate. But with respect to its adaptation to the purposes of navigation, the questions of detail become more complicated. It frequently happens that the transports only require to be effected in one direction, and that they can only be effected under certain conditions of velocity and depth. The width to be given to the new navigable channel may also depend upon circumstances extrinsic from those of the river itself, so that a careful examination of the commercial relations of the district is as necessary as that of its physical nature.

When it is possible to obtain, either artificially or naturally, a depth of about 3 feet, a river becomes navigable for barges. If the rate of fall in the longitudinal direction exceed from 7 to 8 in 10,000, the barges can only descend loaded. It is usual, however, to regard an inclination of 1 in 2,000 as the maximum which admits of transport in the two directions of ascent and descent. The river Rhone has an inclination of from 7 to 8 in 10,000, as quoted above, and by the aid of a class of steamers constructed especially for that river, with some peculiar arrangements of their machinery, the ascending navigation is carried on with

tolerable success. But on the river Lys, in Belgium, where the haulage is performed by horses, the rate of flow, produced by an inclination of 1 in 2,000, might render the ascent difficult, were it not retarded by the aquatic plants, which it is strictly forbidden to cut.

Barges naturally vary much in their dimensions, according to the nature of the river upon which they are employed. The extreme limits of variation appear to be, in length, from 50 to 230 feet; in width, from 6 feet 6 inches to 23 feet; and in draught of water, from 2 feet 6 inches to 6 feet 6 inches. Evidently, then, it is important to ascertain the dimensions of those frequenting the waters of the main stream, or of any of its affluents, before commencing any works for the improvement of either the former or the latter.

In many instances it will be found sufficient for all ordinary purposes of navigation to regularise the outline of the bank nearest to the thalweg, so as to secure a uniform depth of water, and a freedom from abrupt changes of direction, in the part of the channel close to this bank. The towing-path would then, naturally, be formed on the same side; but it is perhaps as necessary to lay down as a general rule, that a towing-path ought to be formed upon the bank under the prevailing wind. The conditions really affecting the determination of its position are that the haulage take place in as direct a line as possible, and that there be very few impediments to the passage of the ropes. It may occasionally happen that a second towing-path is required, but, generally speaking, in these cases the width need not exceed one-half of that of the principal path. Both of them should be kept at such heights as to allow of their being above the water line, so long as the navigation can be safely carried on; directly, however, the waters of a river rise to such a height as to cause the river to flow with a dangerous velocity, it is advisable that they become submersed, in order effectually to prevent the bargemen from attempting to proceed.

The width of a towing-path is usually from 12 to 18 feet; mooring-posts are required on the opposite bank. In passing under bridges the towing-path should be carried under the land-arches, if possible, so as to obviate the necessity for detaching the tow-ropes. When it is not possible to carry the path in this position, it will be necessary to insert rings into the masonry of the bridge, or to place mooring-posts in the banks, or to adopt some other method of attaching the boats during the period that the tow-rope is being carried forward.

But, in the majority of instances, it is necessary to do far more than merely construct towing-paths. The depth of rivers in the summer months is usually insufficient to allow the continuance of navigation; in other seasons the velocity may be too great; sometimes the thalweg may shift from one side to the other, or the banks may be exposed to be frequently washed away. The first object to be obtained is, then, to maintain the river in its bed, and to create for it a channel of such dimensions as to ensure, at the lowest waters, sufficient width and depth; and the second, to regulate its velocity so as to ensure favourable navigation in either direction. They may be obtained, either by forming a series of reaches of still water in the bed of the river itself, communicating with one another by locks; or by means of a lateral canal; or occasionally by constructing a secondary bank, submersible whenever the waters rise above certain definite levels.

If the river flow under such circumstances as to form a succession of islands dividing its waters into two or more branches, advantage may be taken of this circumstance to divert into the main or navigable channel the waters usually flowing in the subsidiary branches, by means of submersible dams or by movable barrages. It is also possible to convert the main channel into a canal, by forming a lock at the extremity of such a series of islands, and placing waste weirs

upon the small communicating channels between them. Such a lock will also, of course, increase the depth of water reserved for navigation, and destroy any injurious velocity of the main stream; but even when it is found inexpedient to construct any works of this class, the fact of confining a stream within a regular channel, and thus concentrating its scouring action, will render great service by eventually lowering the bed of the river.

If the river suddenly diminish in depth on account of the widening of its bed, it may be improved by contracting the latter; the manner to be varied according to local circumstances. Thus, in the case of the Midouze, a river falling into the Adour, in the south of France, the widening out of the channel in several of its bends was corrected very successfully by planting aquatic trees, such as willows, osiers, &c., upon the banks, so as to leave a clear, regularly outlined water-way, at the same time that all reefs or other projections in the channel were removed. When the velocity of the stream is small, this system appears to answer very well; because the artificial banks thus formed soon become raised by the deposition of any mud or sand in suspension in the waters of floods, which is facilitated by the retardation of their flow in consequence of the obstacles formed by the trees, and as the roots of the willow tribe strike quickly they soon solidify the deposits. As the water-way becomes also thus contracted, there is at the same time created a tendency to lower the bed of the river and to cause the water to flow permanently in the open channel thus left. There is a great simplicity in the means adopted in this case; the materials employed are inexpensive and easily procured, and they possess this advantage, that they do not in any way interfere with the normal régime (or conditions of flow as to volume and velocity), in the open channel, at least.

In rivers exposed to sudden and violent floods, however, the trees, and the deposit around them, would be inevitably

carried away; and it becomes necessary to construct the longitudinal banks required to concentrate the summer or low waters in a more substantial manner. But, in such cases, it is equally necessary to defend the usual or natural bank of the river by some system which shall enable it to resist the abrading action of the current. On many rivers, instead of constructing longitudinal banks, a series of transverse spurs are carried out into the stream, for the purpose of giving rise to corresponding pools of still water in which the silt or gravel carried down may be deposited. These spurs, in fact, are intended to exercise the same influence upon the stream of rivers that groins do upon the currents of the sea-shore. In both cases, however, their usual effect is very questionable; that is to say, when compared with their cost. Unless they are in close proximity to one another, the counter-current produced by these spurs is found to corrode the banks in a serious manner on the down side of the projection. If they are very close together, their developed length will be found to be nearly equal to that of the more logical system of longitudinal banks; and in the latter case, moreover, there is less danger to be apprehended from the changes of direction produced by the irregular interferences with the line of the current. Experience appears to warrant the assertion of the general rule, that the most effectual method of deepening the bed of a river, and of regulating its flow, is to confine it between longitudinal banks, which may occasionally require lateral openings or waste weirs, so as to allow any sudden freshets to escape directly they attain a dangerous height.

The submersible banks, or dykes, as they are sometimes called (and the name will be retained for the purpose of designating more clearly the difference between the dykes and the banks), may be executed either in rough blocks of stone, or concrete, of masonry, of woodwork, of fascines, or of panniers filled with gravel or with rubble-stone. The feet of the banks may also be protected in the same manner; and if

any portion extend much below the permanent water-line, a combination of several of the above systems may be employed, as in the case of the banks of the Rhine. In almost every case, however, the determining motive in the choice of materials is to be found in their relative cost. As the two classes of works, viz., those for the defence of the banks and the construction of the dykes, are so nearly identical, the description of the former, by far the most important, will be given in the greatest detail.

The rubble facing of river banks may be resorted to when stone is plentiful and at a very low cost, for it is to be

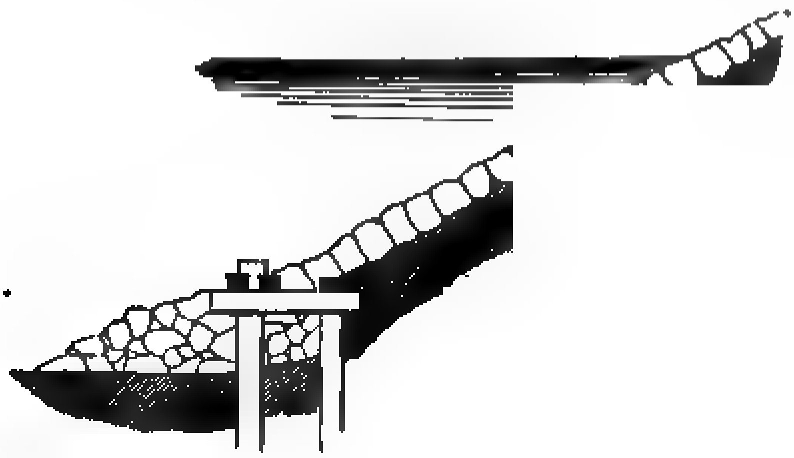


Fig. 122. Rubble-facing, River Banks. Fig. 123.

observed that the quantities required are very considerable. One peculiar advantage of this system is that the rubble easily slides down into any place where the water has attacked and undermined the banks; and it may be executed under almost every condition of the level of the water in the river. The largest stones which it is possible to obtain ought to be employed, because they are displaced with the greatest difficulty. The slopes, when finished, should be dressed tolerably smoothly to a minimum inclination of 1½ to 1; the thickness must depend upon the nature of the bank to be supported, and the degree of erosion it is required to compensate.

When stone is dear, the slopes may be pitched in the por-

tion above the usual summer level; this pitching, however, does not support the banks, but only serves to defend them against any erosive action of the currents or descending ice. In the execution of this pitching the most important part is to be found in the foundations, which must be able to resist the undermining effects of the current. If the bed and sides of the river be of a solid nature, loose rubble may be employed; but if they be of a nature to yield easily, it may be necessary to defend the feet by a single, or even by a double, row of piles, Figs. 122, 123; the woodwork being kept as low as possible in all cases. The inclination may vary from 1 to 1, to 2 to 1; the longer slopes requiring a less thickness of pitching, and resisting the action of the currents more effectually; at the same time they will be found to carry waves to a higher point, if the river be sufficiently wide to allow of their formation. The thickness will be regulated by the rate of inclination and the force of the currents; but it is usually from 9 to 14 inches at the summit, and increases at about the rate of 1 inch to every foot of additional depth. In order to resist the action of the currents at the water lines, the courses should be inclined, or, at any rate, they should not preserve their horizontality for any great distance.

Fig. 124.—Timber-facing, River Banks.

Slopes may be protected from the effects of a sudden flood, when rapidity of execution is desired, by means of a timber facing, as in Fig. 124. Guide piles are driven, either vertically or in an inclined position; they are connected at the top by wales, and on the inside they are lined by planks laid horizontally, and backed by earth. This method of defending banks of rivers is very costly, and can only be employed

definitively in positions where timber is plentiful, land valuable, and the length to be protected not very considerable. It has been adopted on the banks of the Scheldt, and in some of the Atlantic cities of the United States. On the banks of the Upper Po, in the Piedmontese dominions, a very economical system of temporary wooden defences was introduced by an Italian engineer, of the name of Magistrini, which has answered very well in every instance where it has been employed, for the purpose of turning aside any current

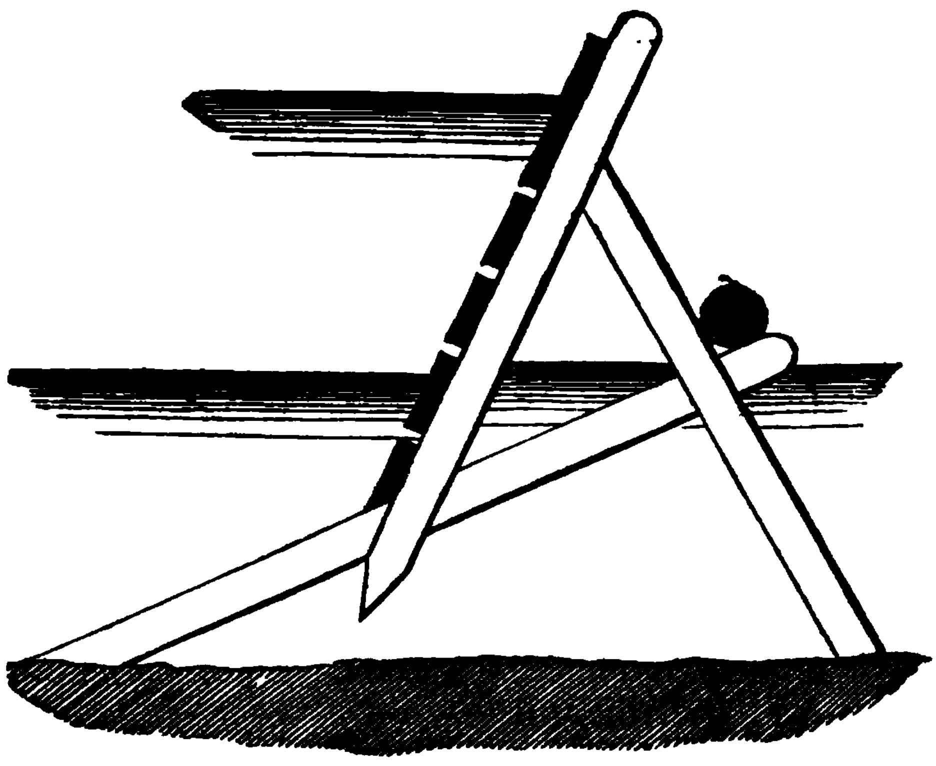


Fig. 125.—Timber-facing, River Po.

acting upon projecting spurs or abrupt elbows on the river banks, Fig. 125.

Fascines are formed by tying together a great number of small twigs of brushwood, laid longitudinally, by other twigs placed at intervals varying with the diameter. The wood ought to be from five to six years' growth, the small and large ends being respectively kept in the same direction, and at least two-thirds of the total quantity used in a fascine should pass from one end to the other, nor should any twig exceed 4 inches in diameter. Small fascines are from 5 feet to 6 feet 6 inches long, and about from 1 foot 6 inches to

3 feet 6 inches in girth at the large end. In Flanders and Holland the length is usually made from 8 to 18 feet, and the girth in the centre from 1 foot 4 inches to 1 foot 8 inches; and upon the Upper Rhine the length is made from 13 to 16 feet, with a girth of from 3 feet 6 inches to 5 feet 6 inches at the larger end, and from 1 foot 8 inches to 1 foot 10 inches at the smaller. Sometimes the fascines are tied together at the ends, so that the small extremity of one may join the large extremity of the other, and the name of "sausage," or "gabion," is given to the assemblage. Military engineers make their gabions about 20 feet long, and 3 feet in girth on the mean, with bands at distances of from 1 foot to 1 foot 6 inches; in Holland the gabions are made from 24 to 27 feet long, from 1 foot 4 inches to 1 foot 8 inches in girth, and with bands at every 6 inches apart; and upon the Upper Rhine the gabions are made from 2 feet 6 inches to 3 feet 4 inches in girth.

In Flanders and Holland, when a bank is to be protected by fascines, if the corrosion take place above the ordinary water line, and the natural slope of the ground below be such as to support the weight of the bank, the fascines are laid in horizontal courses, with the small end towards the land and the butt end to the water. The ends of every succeeding layer are set back from the line of the layer below it, so as to form a regular batter, and the whole body is tied together by means of stakes 4 feet long driven through each fascine as it is placed. The heads of these stakes project from 6 to 8 inches, and they are tied together by hoops passing alternately in and outside of the heads. Gravel, clay, sand, or shingle are then firmly rammed upon the fascines, so that their surface becomes perfectly level before proceeding to place another layer. The batter of a slope thus built up in fascines may vary from $\frac{1}{3}$ and $\frac{1}{2}$ to 1 base to 1 in height.

When the bank is corroded below the ordinary water line,

the course usually adopted is, to form a species of raft of gabions strongly tied together and fixed into the banks by stakes, with their ends projecting into the stream. Other gabions are placed upon these in a direction parallel to the bank; and fascines alternately crossing one another in the body of the raft, but presenting always at their river end their smallest extremity, are laid upon this description of grating. The several layers of fascines are joined together by stakes, round which bands are placed as before, and the whole structure is sunk by being loaded with gravel or stones, forming, in fact, a species of elastic matlass adapting itself to the form of the river-bed. The force and velocity of the current modify to a certain extent the resistance of the gabions; and it is for this reason that in the districts of the Upper Rhine the dimensions given to the gabions are greater than those adopted where the river runs more sluggishly.

Upon the banks of the Rhine panniers filled with gravel are occasionally employed. They are formed of osiers or willow-twigs woven together in the form of baskets, about 6 feet 6 inches long by 3 feet 4 inches high and 2 feet wide, when rectangular; the length is made about 7 feet when the panniers are triangular, the sides measuring 4 feet 4 inches; when they are circular, the length is made 10 feet, and the girth about 7 feet. These panniers are thrown down, sometimes at random; at others they are sunk over the positions they are intended definitively to occupy, and fastened by means of stakes. In several instances, large hollows in the banks of rivers in these districts are filled in with panniers of the above description, and the upper surface is further protected by means of a pitched stone slope laid in the usual manner.

The use of fascines can, however, only be recommended in countries where more durable materials are extremely expensive, or where great rapidity of execution is required. They are exposed to very rapid decay, and to the attacks

of numerous animals. In England they are hardly ever employed by other than the Royal Engineers; and it is in Holland only that they are habitually employed in the formation of longitudinal submersible dykes for the regulation of the channels of rivers. Whatever description of material be adopted, the most important point to be observed is, that the work for the defence of a bank should be executed with the greatest rapidity, and before the corrosions can attain any dangerous extent, which might allow the formation of any secondary branches, or any permanent alteration in the régime of the river.

In all works intended to improve the navigability of a river, extreme care is required in attempting to alter its natural conditions, whether of width or of depth, because the results of any interference with them are always very uncertain. It is preferable at all times to maintain the river within the limits nature appears to have traced for it under its normal flow, rather than to endeavour to introduce great modifications, even though they may appear highly desirable. Thus, in several instances, when it has been attempted to shorten the course of rivers by diverting them into new and straighter beds, the water has eventually formed for itself a course of precisely the same character as the original one. This is particularly true with rivers that run upon a sandy bed, when they follow a sinuous line, and it is attempted to shorten the distance between any of the principal beds; for the slightest inequality in the resistance of either the bed or of the banks will give rise to currents such as are able to overthrow the new works. It may be laid down as a law, that the straightening of the bed of a river is only to be effected to a limited extent, one which will depend upon the nature of the materials over which it flows and the volume of water it carries.

It may be possible to improve the navigability of a river either by dredging or by the closing up of the secondary

branches, without resorting to the construction of submersible dykes; and at almost all times those operations are productive of advantage, if executed with proper precautions. Thus, dredging may be resorted to for the removal of any shoals in that portion of the course of a river where the navigation is designed to take place. But in some cases, as, for instance, when the water is kept back in a series of pools by means of such shoals, their removal may be attended by the lowering of the water line in all the upper parts of the river. The nature and mode of formation of such shoals must therefore be carefully ascertained before an attempt is made to remove them.

In closing the small branches which run between the subsidiary islands so frequently to be met with in rivers, the course usually adopted is, to carry out the dam from either side towards the centre. In proportion as the dam advances the waterway becomes contracted, and naturally the velocity is increased to such an extent as to render the closing of the central portion a very difficult operation, because the bed is deepened, and the materials thrown in are often carried away immediately. Sometimes the aperture is closed by driving piles in front of it, between which hurdles are placed so as to diminish the current; sometimes sheet piling is driven; and occasionally old boats or pontoons, laden with earth or stones, are sunk. Under all circumstances, it is necessary that the materials required to carry the dam up to its full height should be prepared beforehand, so that no opening should be left through which the water should flow. Loose rubble-stone dams are used in some places, and they become eventually watertight by the deposition of the mud and silt brought down by floods. Fascines are used in other cases, either by being thrown down at random, or by being sunk in large rafts in the manner adopted in Holland and Flanders.

The best position of the dam closing such small branches

is a subject of some obscurity. When it is placed at a considerable distance below the point of bifurcation, the water becomes stagnant before arriving at the dam, and deposits the silt required to render it watertight. When it is placed close to the point of bifurcation, the currents and floating ice are likely to damage its construction; and, moreover, the length in such positions is considerably increased. The only general rule appears to be, that the dam should be placed sufficiently near, but below the extremity of the island, to allow the alluvial deposit to reach it easily.

A very important observation is to be made with respect to the diversion of the waters of secondary branches into the main stream; namely, that the depth of the latter will be but slightly increased if the river flow freely, because the volume of water discharged increases more rapidly than the height. Observations upon the meeting of rivers pointed attention originally to this law, and theoretical reasoning confirms it. Some interesting facts connected with this subject will be cited hereafter; but it may be now observed that the effect of any dam or dyke which offers an obstacle to a current is to produce a contraction of the waterway, causing the plane of the water to rise on the up side and to lower on the down side. The inclination of the surface in the portion so contracted is naturally increased, and it is possible that it may be so to such an extent as to interfere seriously with the navigation. If the bed of the river itself be of a light and easily-moved nature, it is equally possible that the increased velocity of the current may deepen it; but, under ordinary circumstances, the materials thus removed are only displaced, for they are deposited in the lower part of the course, where the river in fact begins again to widen out to its natural dimensions. The practical rules usually admitted by engineers in works connected with the narrowing of rivers for the purpose of increasing the depth may be briefly stated to be as follows :—

1. The longitudinal dyke should never be carried to a height above the mean level of the water in the river; the usual height above low summer water level is 2 feet; during floods they should be entirely under water. 2. The velocities of the different channels vary in the inverse ratio of the cube roots of their widths. 3. The cubes of the heights are in the inverse ratio of the widths of the beds. 4. As much as possible it is advisable to preserve the natural waterway, and to make the capacity of the narrowed pass equal to that of the ancient bed in such portions as are free from irregularities. If it be necessary to displace the river, it should be directed, in preference, to the bank or portion of the channel most likely to yield to the scouring action of the waters.

The attention of engineers was called to the peculiar phenomena connected with the junction of rivers by a very remarkable letter or report by Gennété, an Italian engineer, addressed to M. de Raet, burgomaster of the city of Leyden, about the year 1755. In this letter he showed that a large watercourse could receive all the water brought into it by an affluent of considerable volume, without any sensible augmentation of the height of the water line, or without any increase in the width of the bed. The reason he assigned for this was, that at the same time that the volume was increased the velocity was augmented in such a proportion as to compensate for the effects it would be natural to expect under the above circumstances. At the same time he showed that the formation of new channels would not lower the water line in any main stream, but only retard its velocity. He cited in confirmation of his views the facts that the Danube, after receiving the Inn, neither increased in width nor in depth; nor did the Rhine occupy a larger bed in passing through Cologne, after receiving the Moselle, than it did in the upper part of its course. On the other hand, he observed that the Rhine, near its embouchure, divides into the several branches of the Waal, the Yessel, and the Rhine proper, without any

sensible depression of the water line either in the parent stream or in the branches. It may be interesting to observe here that it has been observed also in America, that the width of the Mississippi after the junction with the Ohio is even less than that of the former river above the point of junction.

A cursory examination of a map will show that the law which appears to regulate the formation of the deltas or alluvial deposits at the mouth of all great rivers is, that these divide into a series of subsidiary branches before falling into the sea, and that they almost always project beyond the line of the sea-shore, forming, as it were, projecting promontories of a rounded outline connected with the original line of coast: the Volga, Danube, Rhine, Rhone, Po, Nile, and the Mississippi, Ganges, Irrawaddy, &c., may be mentioned as illustrations. The lands around these deltas are flat and marshy, and consist of sand and mud; the channels winding through them are shallow, and exposed to change in their direction and volume without any apparent cause. The rate of deposition depends upon circumstances equally beyond ordinary calculation, and varies in every particular river.

The formation of these deltas arises from the deposition of the matters brought down from the upper portions of the river courses, caused by the difference in the specific gravities of the fresh and salt waters, and the annihilation of the onward movement of the former by the tides or the littoral currents of the latter. The tendency to the deposition of alluvions is also increased by the diminished inclination and velocity of the rivers near their mouths, and in some cases this diminution is sufficient to cause the rivers to overflow the lands above the delta itself, so as to render it difficult to ascertain its precise limits. The formation of the subsidiary branches noticed in the deltas is to be attributed to the reflux of the waters in the main channel, and they are

the most numerous in the deltas which advance with the greatest rapidity and occupy the greatest areas. But if the littoral current run with great velocity and transport any alluvions, it will give rise to bars at the mouth of the branches; if it be free from alluvions, it will disperse the materials it may detach from the extremities of the deposit, or, deflecting the line of outflow of the river, it will give rise to a delta following the direction of the resultant of the two forces. Occasionally also the tidal waves will give rise to a bar across the mouths of the branches at the points where they begin to neutralise the outward flow of the soft water.

The majority of the English rivers fall into the sea at the bottom of large open bays, and in those cases the deposition of the fresh and salt water alluvions takes place in the form of banks or shoals in those portions of the bay where the respective currents of the sea and of the river meet. These shoals vary constantly in their outline and position in such rivers as the Thames and the Severn, and in the Seine, Loire, and Garonne in France; and form very serious impediments to the navigation. In many other cases, as in the rivers upon the Suffolk and Norfolk coast, and upon the southern shores of England, the deposits take place across the mouths of the rivers, according to the law noticed in the last paragraph. But it is to be observed that, although these bars diminish the depth of water immediately over them, the river above may often retain a very considerable depth; indeed the effect of the bar is often precisely analogous to that of a dam. Thus, the Rhone has rarely a depth of more than 6 feet 6 inches in the passes of its delta, whilst at Arles the depth is not less than 43 feet. At the mouth of the Po di Volano the pass has only a depth of 2 feet 6 inches, whilst about seven miles further up the depth is not less than 10 feet; and the same fact has been observed at the mouths of the Nile and of the Mississippi, but to a far greater extent.

At the junction of streams in the interior, the same phenomena may be observed to take place as those which have been already noticed as occurring on the sea-shore, although of course upon a very diminutive scale. Should one of the confluents bring down much alluvial matter, and flow with considerable velocity into a stream of a different character, the deposit may take place either in the fan-like shape of a delta or as a bar; and in the former case it is possible that the stream may divide into a number of branches, whilst in the latter the relative depths of water over the bar and above it may present all the essential conditions of those connected with rivers discharging into the sea. But the absence of the tidal action gives a greater fixedness and simplicity of character to the manner in which these deposits are effected, and consequently allow of their being treated with greater comparative facility, whenever it is desired to improve the navigation of a river previously obstructed by them. But it must always be borne in mind, whether it be a question of combating the natural operations of the laws affecting the flow of large or of small streams, whether strictly inland or in estuaries, that unless some other natural law of the same class be made to counteract the particular one producing the state it is desired to remedy, all engineering contrivances or mechanical operations will either be vain, or at most produce but a temporary effect. Nature must, in fact, be made to correct itself.

If, therefore, it be desired to obviate the inconvenience arising from the deposition of the alluvial matter across the embouchure of an affluent into a greater stream, and this deposition be found to be caused by the annihilation of the velocity of the affluent, in consequence of the greater velocity of the main stream, the only effectual mode of proceeding would be, to increase the velocity of the affluent by diminishing its bed; or perhaps, if it follow a devious course, by increasing the fall by means of a new and shorter channel.

It would also, in some cases, be possible to effect the desired object by forming an artificial embouchure at some other point on the river, where the régime of the main stream might be very different. But whatever precise details be adopted, they must resolve themselves finally into the means of securing an equality of velocity in the confluent rivers, and so directing the respective axes of their flow as to prevent the stream of the one from setting across the line of the other.

Should the deposit assume the form of an ordinary delta, and give rise to a subdivision of either or both the rivers into a series of small branches, the best course to adopt is, to form new beds for the waters brought down, of such sectional area and inclination as to insure the meeting of the respective streams under those conditions which would allow the alluvial matters to deposit themselves upon lines of direction corresponding with the resultant of the two new channels. As the tongue of land thus formed would continue to advance, it would be necessary to provide for the inevitable changes it would superinduce upon the point of junction.

On the sea-coast, if it be desired to lower the water line in the branches of a delta, Gennété's experiments, and all subsequent experience, show that the most efficient method of proceeding is, to increase the velocity of flow by causing a greater volume of water to pass through a given channel in the same period of time, rather than by increasing the surface of the waterway. This is especially the case when the bottom of the river is composed of a material capable of being easily removed, because the bed itself will be lowered in consequence of the increased transporting power of the water. Evidently then, in such positions, the width of the subsidiary channels is the only element which is susceptible of modification by artificial means, at least economically, and it should in all cases be reduced as much as possible. All the small branches which can conveniently be suppressed

should be closed, so as to concentrate all the action of the water upon the channels it is proposed to retain for the purposes of navigation. These remarks are principally made with reference to tideless seas; the difference in the system to be adopted where the range of the tides is considerable is substantially unimportant, and will be mentioned hereafter.

In the case of a bar formed across the mouth of a river on the sea-shore, human means are almost powerless if the littoral current bringing the alluvions be strong, and not susceptible of being diverted. To a certain extent it is possible to augment the depth of water above the bar by concentrating the outflow of the river, and guiding it by means of parallel banks in such a manner as to direct the scouring action of the land water so that it should remove the alluvions either in the deep sea or again into the littoral current which should carry them further on. But the success of such measures will only be temporary, and the history of the ports of Rye, Dunkirk, Aigues-Morte, and others, show that it is hopeless to endeavour to struggle against the ceaseless unwearying operations of nature. New channels may be carried out through a bar, and for a time kept clear by the scour of the upland waters and by dredging; but sooner or later the angle between the original outline of the bar and the projection of the new channel becomes filled in, and if the littoral current should not then be able to sweep off the alluvions carried to the front of the pass, the bar will begin to reform. The pass or channel must then be carried further out to sea, or kept open artificially at a constant and excessive cost. It is possible to direct the tidal action in some positions, so as to produce considerable modifications in the ordinary laws affecting bars, in the following manner, which is applicable to either of the cases connected with the entrances to rivers hitherto considered.

In the lower zones of rivers exposed to tidal action, the depth of the navigable channel may be increased by causing

a larger volume of water to enter with the flood, and confining its outflow during the ebb within the limits of the channel. If then a river divide into several branches, the subsidiary ones may be closed by means of dams, with sluices opening only with the flood, and so arranged that the water thus introduced into the secondary channels shall be forced to escape by the one it is proposed to deepen. This object may also occasionally be effected by causing the secondary channels to derive their supplies from the principal one at some point on the up stream; but it will generally be found that in such cases the velocity of the ebb tide will be inconveniently great for the ordinary purposes of navigation. In fact the secondary channels would become sluices, whose waters would act to scour the pass. The advantage proposed to be gained in these instances is to be found in the fact that a large quantity of water is forced to flow through a narrow channel, and the velocity is consequently increased, thereby enabling the river to remove the lighter materials of which its bed is composed. But the whole success of works intended to produce these results must depend upon the relative proportion of tidal water which can be made to enter, or upon the increase of velocity in the outflowing stream. It follows, therefore, that any diminution of the water surface at high tide is likely to affect seriously the power of a river to maintain a clear navigable channel, unless the latter be contracted at the same time in a corresponding degree.

The results obtained at Nieuwe Diep, upon the Clyde, the Dee, the Thames, and the Seine, confirm what has been above stated; and, moreover, the practical deductions to be made from them appear also to warrant the assertion that it is preferable to obtain the increased scouring action of the tide by forcing the water to flow higher up the river, rather than to allow it to spread near the embouchure. The works of Nieuwe Diep have already been described; those upon

the Clyde and the Dee and the Ribble have consisted in the erection of longitudinal dykes parallel to the axis of the river, retaining the stream within a narrow channel during the latter portion of the ebb, and in the removal of obstructions to the propagation of the tide wave to a greater distance from the embouchure than it had previously obtained. In the Thames, the removal of the Old London Bridge, which acted as a dam to the tide, has been attended by an increase in the duration of the flood, and an increase of depth at high water varying from 1 foot 8 inches at Teddington to nearly 7 feet at Blackfriars Bridge. In the Seine, the concentration of the tidal action, by means of the lateral banks lately formed between Caudebec, Villequier, and Quillebœuf, has deepened the river a little more than 9 feet, partially destroyed the bore, and increased the duration of the flood tide one hour. Nor does there appear to be any reason why these results, so advantageous to the interests of commerce, should not be further developed, if the existing obstacles to the propagation of the tide wave into the interior were removed upon the two last-named rivers.

Precisely opposite results have attended the diminution of the scouring reservoirs naturally existing beyond the channel of the port of Ostend. The marshes or low lands, there flooded at every high tide, have been gradually reclaimed, and as the channel was not carried further up into the country, so as to create an artificial backwater whose conditions of discharge should replace those under which the waters over the low lands escaped, the silt brought into the mouth of the harbour by the littoral current has considerably diminished the depth in the entrance. Dredging and sluicing have been resorted to in vain, although conducted with all the practical skill and persevering energy of the Dutch and Belgian engineers. For, however powerful the effects of sluices may be, they are far inferior to those of the alternate

currents of the flood and ebb tides spreading over large spaces. Great circumspection must therefore be exercised, and long, elaborate, and skilful investigation made, before any port or river is deprived of the scouring action of the tides. The alluvial deposits may perhaps tend naturally to diminish or to destroy this action; but it must be retained as long as possible, and our efforts directed at all times rather to increase than to diminish its power.

[Mr. D. Stevenson classifies the upper, middle, and lower sections of rivers as three compartments. 1st. The seaward compartment, or the "sea proper," the estuary, in which the tides resemble the tides of the adjoining sea. 2nd. The tidal compartment, in which the phenomena of ebbing and flowing take place, but in which the times of ebb and flow do not remain constant, the time of the ebb gradually gaining upon that of the flow, and the duration of low water is gradually protracted in proceeding up the river until the influence of the tide vanishes. 3rd. The river proper compartment, the characteristic of which is the total absence of ebbing and flowing, the river at all times pursuing its downward course in an uninterrupted stream and at an unvarying level, except in so far as it may result from the changes due to land floods.

Noticing, first, the uppermost section of rivers, the engineering works may be said, in general terms, to consist chiefly of weirs built across the stream, by which the water is dammed up and forms stretches of canal in the bed of the river, with cuts and locks connecting the different reaches. In this manner, according to Mr. Stevenson, the river Schuylkill, in Pennsylvania, was rendered navigable. Thirty-four dams were constructed in the bed of the stream, so as to raise the level of the water and convert the river into 34 reaches of navigable water, varying in length according to the rise in the bed of the river. The barges pass from the

successive reaches through a short side cut at the end of the dam, in which there is a canal lock of the ordinary construction. The navigation is upwards of 100 miles in length, traversed by boats of 60 tons burthen.

The same plan has been followed in the improvements of the river Lea by Mr. Rendèl and Mr. Beardmore; also by Sir William Cubitt on the upper part of the river Severn, where the river has been divided into four reaches, having a

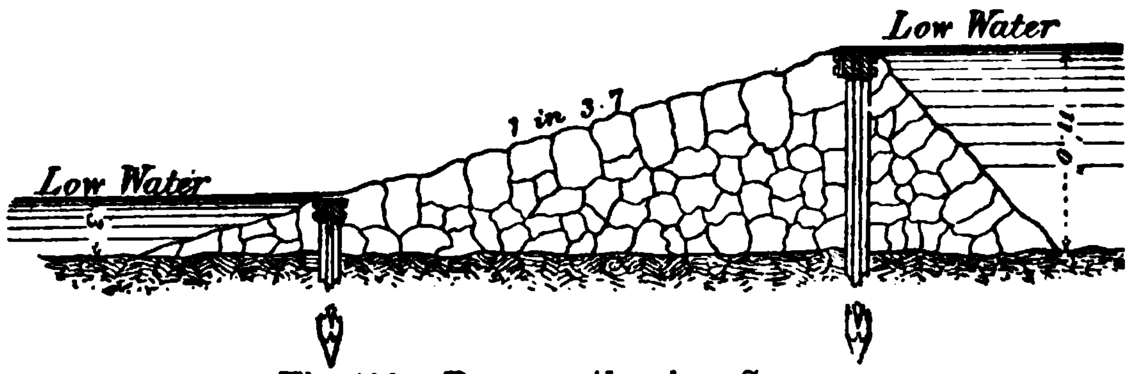


Fig. 126.—Dams on the river Severn.

depth of 6 feet, with side cuts and locks having a lift of 8 feet each.

The dams or weirs constructed for arresting and raising

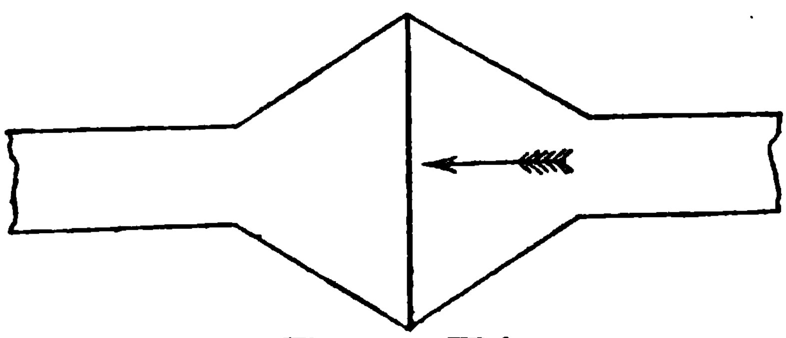


Fig. 127.—Weir.

the waters of rivers are works which demand careful consideration in their design. The dams on the Schuylkill and other American rivers are formed of timber frame-work

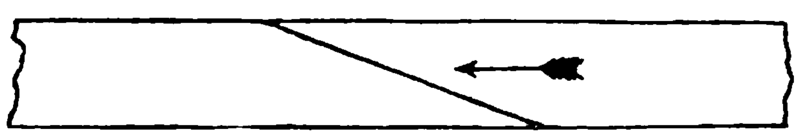


Fig. 128.—Weir.

filled with rubble. The dam at Philadelphia was formed in separate compartments or frames, each of which was 20 feet in breadth of face, and 72 feet deep horizontally.

They were filled with stones, and sunk in the line of the dam. The frames or cribs were constructed of logs 18 or 20 inches square, strongly dovetailed together, covered with planking 6 inches thick. The upper parts of the cribs were connected together so as to form a continuous structure, and the whole dam was backed by a mass of heavy rubble. This dam was strong enough to withstand without injury the force of a flood going 8 feet deep over the crest.

The dams or weirs constructed by Sir William Cubitt on the Severn, Fig. 126, were made of pile-work and rough masonry. He decided to make the weirs of a length equal to three times the width of the channel, and instead of placing them at right angles, as in Fig. 127, he placed them obliquely, as shown in Fig. 128.]

CHAPTER XV.

BRIDGES.—PRINCIPLES OF THE ARCH.

We shall commence with the simplest case, that of two beams, A B and A C, Fig. 129, resting against each other at their upper extremities, and against two walls at their lower, and sustaining a weight suspended from A; and we shall proceed to examine the strains, both on the beams and on the walls, occasioned by this weight. Let the weight suspended from A be represented by the line A D; draw D E parallel to A C, and D F parallel to A B, also E G and F H perpendicular to A D; then, by the principle of the parallelogram of pressures, the strain on A B in the direction of its length is represented by A E, and that on A C by A F. Now, each of these strains may be resolved into two others, one acting vertically upon the wall in the direction of gravity, and the other acting horizontally, and tending to push the two walls asunder. Thus A E may be resolved into the strain A G acting vertically, and E G horizontally; and A F into the strain A H acting vertically, and H F horizontally. Now, the two triangles A E G and D F H being equal, the similar sides H D and A G are equal; and therefore the two vertical strains A G and A H are together equal to A D, or the weight suspended from A; that is, the whole weight borne by both walls is equal to that suspended from A; but

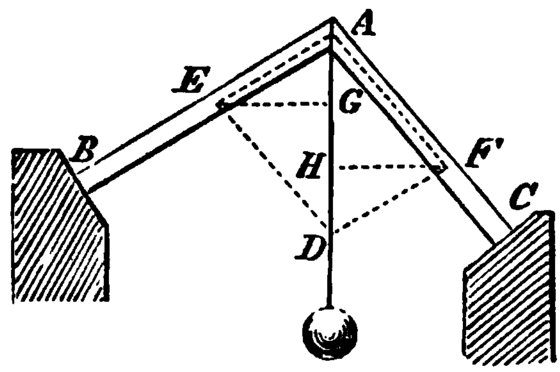


Fig. 129.

the amount borne by each wall depends upon the relative inclination of the two beams. It must further be observed that the lines E G and H F, representing the strains acting horizontally, are equal; that is, whatever may be the relative inclination of the two beams, their horizontal thrust against the walls is the same, and is equal to that with which they press against each other at A.

Let Fig. 130 represent a system of framing, composed of four beams, united together in such a manner as to form a polygon, and so connected at the points C, D, E, F, and G, as to admit of motion about those points; so that the beams are

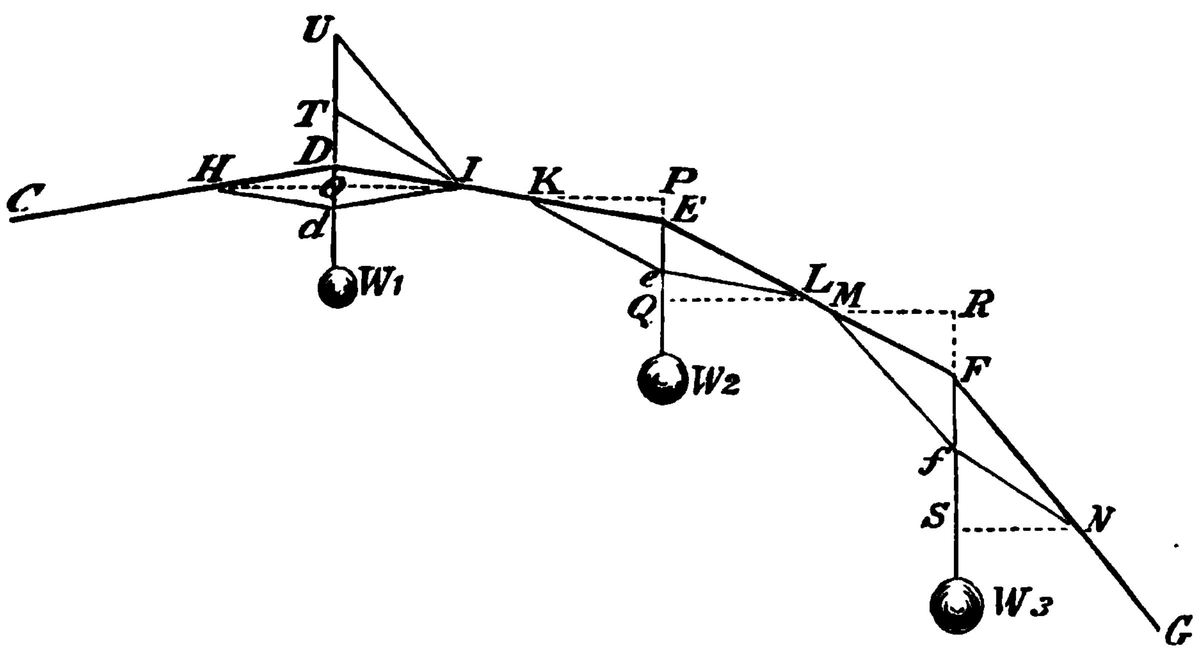


Fig. 130.

not rigidly fixed in the position shown in the diagram, but are free to assume any other form which any external force applied to them might tend to produce. Further, let us suppose weights W_1, W_2, and W_3, to be suspended from the points D, E, and F, and so proportioned to each other that the framing, under the influence of the strains which they produce, is in equilibrium, or has no tendency to alter its form. Let the lines D *d*, E *e*, and F *f*, represent the weights applied at the points D, E, and F, and let each of those weights be resolved into the strains which they produce in the two contiguous beams, by constructing the parallelograms

D H *d* I, E K *e* L, and F M *f* N. Then the lines D I and E K will represent the two strains acting in opposite directions upon the beam D E, and E L and *f* M will, in like manner, represent the two strains acting similarly upon E F. Now, since the whole system is in equilibrium, and all its parts free to move, it follows that each of its several parts, and therefore the beams D E and E F, must also be in equilibrium; such being the case, it results that the strain represented by D I must be equal to that represented by E K, otherwise the beam D E would move in the direction of the greater strain, and, in like manner, the strain E L must be equal to F M. Now, let each of these strains be resolved into two others, acting vertically and horizontally; then will the latter be represented by the lines O I, P K, Q L, R M, and S N. Now, since D I, E K, and *e* L are all equal, and the triangles D I O, E K P, and *e* L Q are similar, the lines O I, P K, and Q L, and therefore the strains which they represent, must all be equal. Again, since E L, F M, and *f* N are all equal, and the triangles E L Q, F M R, and *f* N S are similar, the lines Q L, R M, and S N, and therefore the strains which they represent, must all be equal. *That is, in a system of polygonal framing, whose several parts are in equilibrium, the* horizontal *strain or thrust at all the joints is the same.*

Let us now draw through I the line I T parallel to E F, and the line I U parallel to F G; then, since O I is equal to Q L, and (T I being parallel to E L) the angle E L Q similar to T I O, it follows that T I must be equal to E L. In like manner, S N being equal to O I, and the angle F N S similar to U I O, U I must be equal to F N. Since, then, the lines E L and F N represent the strains on the beams E F and F G, so do also the lines T I and U I; *therefore in a system of polygonal framing whose several parts are in equilibrium, the strains on the several beams*, in the direction of their lengths, *are represented by lines drawn through a given point parallel to those directions, and limited by a given vertical line.*

In like manner, it may be shown that the portions U T and T D of the vertical line U *d*, cut off by the lines drawn parallel to the several beams, are equal to the lines F *f* and E *e*, which represent the weights suspended from those beams.

From the foregoing investigation, we derive an easy method of determining the several strains in any system of polygonal framing whose parts are in equilibrium. Let Fig. 131 represent such a system, kept in equilibrium by the weights W_1, W_2, &c., suspended from its angles; then draw the vertical line L S, and divide it into portions L M, M N, N O, &c., proportional to the weights W_1, W_2, W_3, &c.; through the points L, M, N, &c., draw lines parallel to the directions of

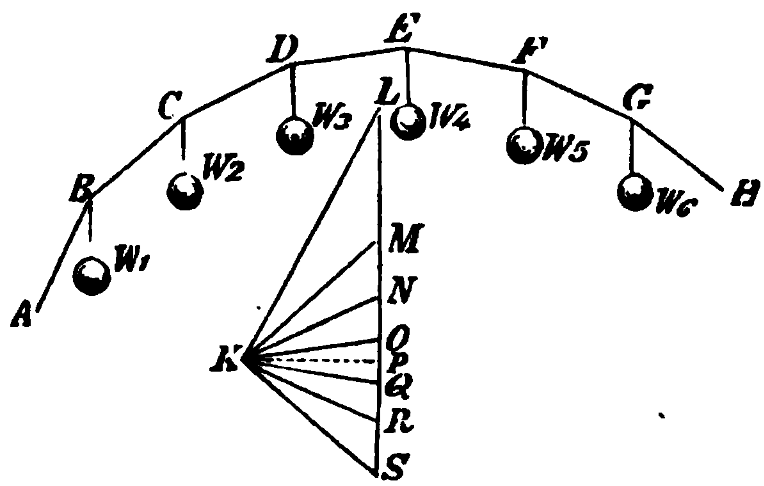


Fig. 131.

the several beams A B, B C, C D, &c.; then, if the system is in equilibrium, all these lines will intersect in the common point K; draw K P perpendicular to L S, then will K P represent the horizontal thrust against each of the joints B, C, D, &c.; the lines L K, M K, N K, &c., the strains in the direction of their length, of those beams which they are severally parallel to; and by the construction, L M, M N, &c., will represent the vertical weights on the several angles, the whole line L S being equal to the sum of all the weights together.*

* If the line K P, Fig. 131, be considered as *radius*, then will the lines L K, M K, &c., be the *secants*, and L P, M P, &c., the *tangents*, of the angles L K P, M K P, &c. That is, if we make the horizontal strain in any system of polygonal framing whose parts are in equilibrium,

Equilibrium of Arches.

The foregoing principles contain all that is necessary to the determination of the equilibrium of arches. An *arch* may be defined to be an assemblage of wedge-formed bodies, the first and last of which are sustained by a support or *abutment*, while the intermediate ones derive support and are sustained in their positions by their mutual pressure, and by the adhesion of cement interposed between their surfaces. The wedge-formed bodies A, B, &c., Fig. 132, thus sustained, are termed *voussoirs*, the centre one D, or that in the highest part or *crown* of the arch, being called the *key-stone;* the inferior surface of the arch E F G is called its *intrados*, or sometimes its *soffit*, but this latter term is sometimes restricted to mean only the under surface of the arch at its key-stone or crown F; the exterior surface H I K is called its *extrados*. The points E and G, where the *intrados* meets the abutment, are called the *springings*, their horizontal distance E G the *span*, and the distance F L the *rise* of the arch.

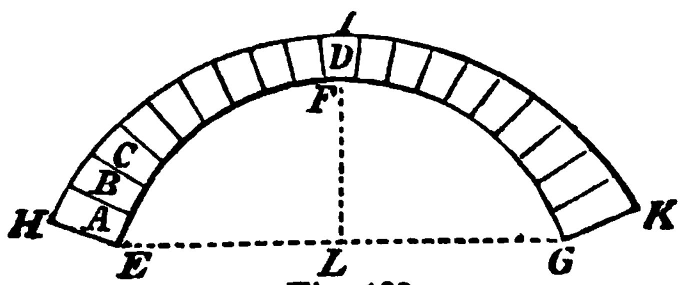


Fig. 132.

Let A, B, C, &c., Fig. 133, be the separate stones or voussoirs of an arch whose several parts are in equilibrium. Now, each stone is acted upon by three forces, namely, the weight of itself and the load above it acting in a vertical direction, and the pressure of each of the two contiguous stones acting in directions perpendicular to their surfaces of mutual contact; then, since these forces must all be in equi-

equal to radius, then will the strain upon any bar of the polygon, in the direction of its length, be equal to the secant of the angle which it makes with the horizontal; and the weight suspended from any joint of the polygon will be equal to the difference of the tangents of the angles which the two bars meeting at that joint make with the horizontal.

librium, their lines of direction must all intersect in some common point within the stone. Let A, B, C, &c., represent these points in the several stones composing the arch shown in the figure; then if lines A B, B C, C D, &c., be drawn connecting these points, they will represent the directions in which the stones press on each other, and the line A B C, &c., is termed the *line of pressure* of the arch. Now, although the pressure of one stone upon its neighbour, as of E upon D, is actually spread over the whole surface of the joint H I, we may, without in any way affecting the question under consideration, suppose the whole pressure collected in the point in which the line of pressure cuts the joint H I, and similarly of all the other stones; so that if we conceive the whole weight of each stone, and of the load which it supports, to be collected in (or, which is the same thing, suspended from) the points A B C, &c., and those points to be connected by inflexible bars, A B, B C, C D, &c. (themselves devoid of weight), we shall in nowise alter or disturb the state of equilibrium of the arch.

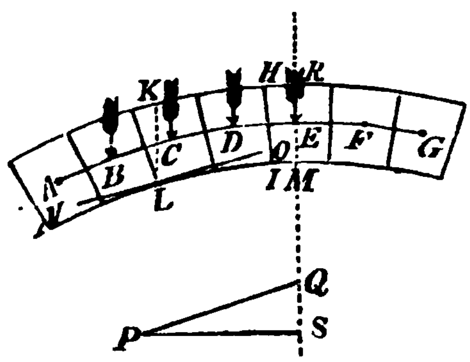


Fig. 133.

An arch, thus considered, is precisely similar to a polygonal framing whose sides are A B, B C, C D, &c., and therefore all the principles which we have deduced in the investigation of the latter may be applied to the arch. This application, however, involves the use of mathematical formulæ and terms which cannot be here introduced, and we must, therefore, content ourselves with stating the conclusions to which they lead; which conclusions are, that for an arch to be in equilibrium, the *vertical* depth of the arch at any point must be inversely proportional to the radius of curvature of the arch at that point, and directly proportional to the cube of the line drawn parallel to the tangent to the arch at that point. Thus, supposing the arch in Fig. 133 to be in equi-

librium, draw the horizontal line P S parallel to the tangent to the arch at the crown M, and through P draw the line P Q parallel to N O the tangent to the arch at any point L; then the vertical depth R M at the crown is to the vertical depth K L at any point L, as the cube of the line P S, divided by the radius of the arch at M, is to the cube of the line P Q divided by the radius of the arch at L.

These conditions are fulfilled in a circular arch, by making the vertical depth D C, Fig. 134, at any point C, equal to the depth of the key-stone A B, multiplied by the cube of the radius A E, and divided by the cube of the vertical height C H of the point C above the diameter F G.

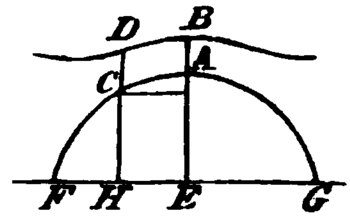


Fig. 134.

In an ellipse they are fulfilled when the vertical depth D C, Fig. 135, over any point C, is equal to the depth of the key-stone A B, multiplied by the cube of A E, half the shortest axis of the ellipse, and divided by the cube of the vertical height C H of the point C above the longer axis F G of the ellipse.

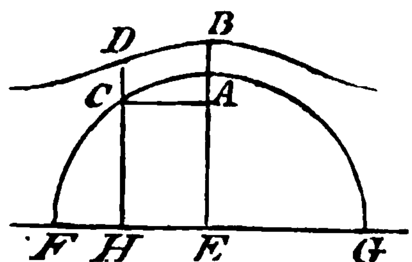


Fig. 135.

The extrados of a circular arch, whose parts are all in equilibrium, may also be determined geometrically in the following manner:—Let B C D, Fig. 135*a*, be half a semicircular arch, whose centre is A, and B E the depth of its key-stone; then in the vertical line A B take the point F, such that the distance D F is equal to A E, and through F draw the horizontal line F G.

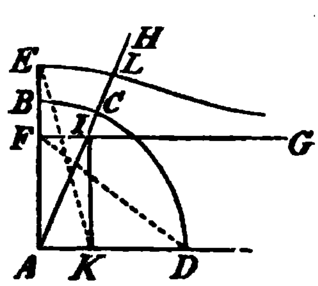


Fig 135*a*.

Then, through any point C, draw the line A H from the centre A, and through I, the point in which it cuts F G, draw I K perpendicular to A D; then make A L equal to K E, and the point L will be a point in the extrados of the arch, any number of points in which may be determined in a similar manner.

An arch in which the above conditions are fulfilled is in a perfect state of equilibrium, every portion of it is equally strained, and no party has a tendency to yield before another. If in an arch so circumstanced, all the joints of the voussoirs were perpendicular to the inner surface, or intrados of the arch, then would the line of pressure pass through the *centre* of every joint, and would cut them all in a direction perpendicular to each. In practice, however, this state of things seldom, if ever, occurs, the line of pressure neither passing through the centre of the joints of an arch, nor being perpendicular to them in direction; it is, therefore, desirable to examine to what extent these conditions may be deviated from without endangering the stability of the structure. When an arch is in a state of perfect equilibrium, if we suppose its abutments incapable of yielding, it can only fail in consequence of the crushing of its material, the cohesive power of which is then the limit of the strength of the arch. When, however, an arch is not in a state of equilibrium, it may fail in two ways: in the first case, the stones may slide upon or slip past each other, and so become displaced; and, in the second case, they may yield by turning upon some of the joints, the arch separating into three or four large portions, as in Figs. 186 and 187, and turning on the inner and outer edges of certain of the joints. Now the voussoirs of the arch cannot slide upon each other unless the angle which the line of pressure makes with a perpendicular to the joint is equal to, or greater than, the *limiting angle of resistance* for the material of which the arch is composed, which is usually about 30° for stone; and as this is very much greater than the angle which the line of pressure ever makes with the perpendicular to the joint, an arch may be considered in no danger of giving way from the *slipping* of its voussoirs; to which we may add, that the adhesion of the cement interposed between the stones, as also the joggles frequently inserted, are an additional security against the

failure of an arch from this cause. The second mode of failure, which is also the most usual, takes place whenever the line of pressure deviates so far from the position of equilibrium as to pass entirely out of the substance of the arch, so as not to cut the joints at all. The more the line of pressure deviates from the centre of the joints, the less will be the stability of the arch; but so long as it continues to cut the joints no motion can take place; the moment, however, that it passes without the joint, motion will take place, the two voussoirs will turn upon their edges nearest to the line of pressure, and the arch will fall. Thus, in Fig. 136, if, by placing too great a load upon the crown of the arch, we alter the form of the line of pressure until it rises above the extrados at B, and falls within the intrados at the points A and C, the arch will separate at the nearest joints to those points into four portions, which will turn upon their inner edges at A and C, and upon their outer edges at B, the arch sinking at the crown and rising at the haunches. If, however, there be a deficiency of weight at the crown, then the line of pressure will fall within the intrados at B, Fig. 137, and rise above the extrados at A and C, the arch separating as in the former case, but now turning about the outer edges at A and C, and about the inner edges at B, the crown rising and the haunches falling in. We see, then, that when we deviate so far from the arch of equilibrium as to cause the line of pressure to approach either the intrados or extrados of the arch, we begin to endanger its stability, actual contact with either being the ultimate limit; and the stability of the arch being greater, as we make the line of pressure approach nearer to the centre of the joints.

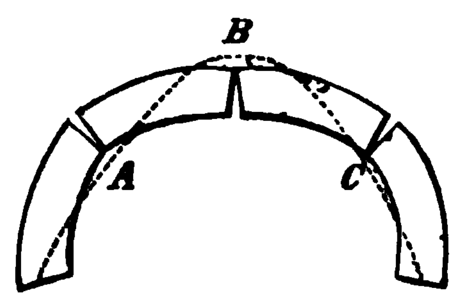


Fig. 136.

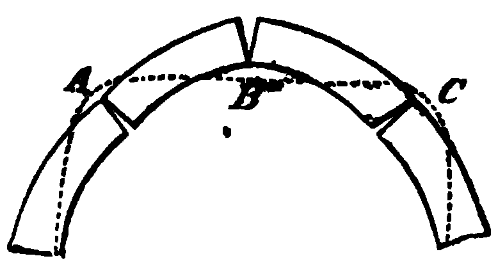


Fig. 137.

When an arch has all its parts in equilibrium, it has been shown, page 211, that the horizontal strain on every joint is the same, and therefore the perpendicular pressure, tending to crush the key-stone of the arch, is equal to the horizontal thrust against its abutment. In order to determine the amount of this strain, let F G H I, Fig. 138, represent the centre voussoir or key-stone of an arch whose centre is A; let C B be the direction of the line of pressure of this voussoir on its neighbour, perpendicular to the joint F H; and let half the weight of the key-stone and the load which it supports be represented by the line B D; then it has been shown that the horizontal pressure on the joint F H will be represented by C D; that is, as B D : the weight on half the key-stone : : C D : the horizontal pressure against the same. Now, the triangle A H E is similar to the triangle C B D, and therefore H E : B D : : A E : C D; further, the weight on half the key-stone is equal to half its breadth in feet, or H E, multiplied by the weight on every foot; also A E is the radius of the arch at the crown; therefore H E : H E multiplied by the weight on every foot of the key-stone : : the radius of the arch : the horizontal pressure against the key-stone; *or, in an arch in equilibrium, the* horizontal pressure on the key-stone *is equal to the weight on a foot of the surface of the same, multiplied by the radius of the arch in feet.*

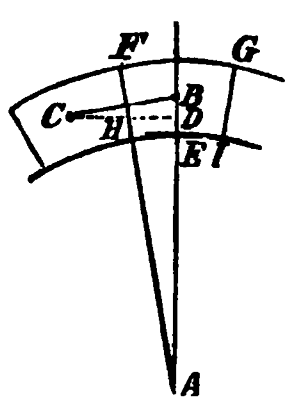


Fig. 138.

The power of an arch to resist the horizontal strain at the crown is proportional to the depth of the key-stone, and to the cohesive power of the material of which the arch is composed. The *stability of an arch* is, therefore, directly proportional to the depth of its key-stone, multiplied by the cohesive power of the material, and is inversely proportional to its radius of curvature multiplied by the weight on every foot of its surface.*

* Let R be put for the radius of curvature of an arch at its crown,

In arches of timber or iron the construction is usually such that the arch is rigid, or incapable of altering its form; and therefore no regard is paid to the direction of the line of pressure, or to the equilibration of the arch. In this case, the arch may be considered as composed of two parts, resting upon the two abutments, and against each other at the crown; and, in order to determine the stability of the structure, it is only necessary to consider the mutual pressure of the two parts against each other acting horizontally at the

d for the depth of its key-stone, and *b* for the breadth of the arch, all in feet; also let *w* equal the vertical weight on every square foot of the key-stone, including its own weight, P equal the horizontal pressure upon the key-stone, and *c* the weight required to crush a square foot of the material of the arch, all in pounds; then

$$\text{P} = \text{R}\, b\, w; \text{ and}$$

the stability of the arch will be proportioned to $\frac{d\,c}{\text{R}\,w}$, which expresses the number of times that the strain upon the arch is less than that which would cause it to yield by crushing at the key-stone.

The following table exhibits the approximate value of $\frac{d\,c}{\text{R}\,w}$ for some of the principal bridges.

Name and situation of bridge.	Radius of curvature of the arch, in feet, at the crown.	Value of $\frac{d\,c}{\text{R}\,w}$ or number of times that the pressure on the key-stone must be increased to crush it.
	Feet.	
Bridge carrying the Great Western Railway over the Thames at Maidenhead .	169	3
Neuilly Bridge, over the Seine, at Paris .	260	5
Bridge of the Holy Trinity, over the Arno, at Florence	172·63	21
Bridge over the Dee, at Chester . .	140	22
London Bridge, over the Thames . .	162	40
Bridge over the Dora Riparia, near Turin	160	44
Bridge of St. Maxence, over the Oise .	121	53
Waterloo Bridge, over the Thames . .	112·5	68

crown, and the pressure of each upon its abutment acting perpendicular to the direction of the same. Thus, suppose A B C D E, Fig. 139, to represent half an iron bridge, G being its centre of gravity, and F G M the vertical direction in which its weight acts; draw F H through the centre of the vertical joint E D perpendicular to the same, and F I through the centre of the springing B C and perpendicular to it; then, in order that the arch may be properly supported, these two lines should intersect in some point F, in the vertical line F M passing through the centre of gravity G; and such being the case, if F G represent the weight of the half arch A B C D E, then will F K represent the pressure acting in the direction F H upon the joint E D, and F L will represent the pressure acting in the direction F I upon the abutment B C.

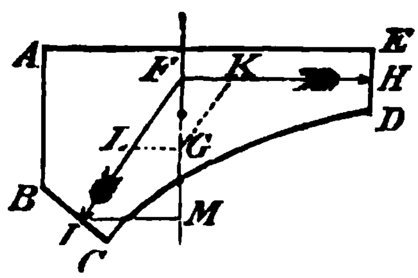


Fig. 139.

Then, by similar triangles, F K : I M : : F G : F M, and I M may be taken as equal to the horizontal distance of the centre of gravity of half the arch from its springing, and F M as equal to the rise of the arch, or the vertical height of its crown above the springing line; *therefore, as the horizontal distance of the centre of gravity of half the arch from its springing is to the rise of the arch, so is the* horizontal thrust, either against the abutment or at the crown, *to the weight of half the arch.*

CHAPTER XVI.

BRIDGES.—SELECTION OF SITE, AND DETERMINATION OF THE KIND OF BRIDGE.

A GREAT variety of circumstances require to be considered in determining the best position, proportions, and materials of which to construct a bridge for any particular situation. The most general and important object to be attained is the establishment of a convenient and permanent means of communication between the two opposite shores, with as slight an interference with the free navigation through the bridge as possible; and the attainment of this object is, in many situations, attended with considerable difficulty, inasmuch as the conditions requisite for the preservation of the navigation are incompatible with those required for the formation of a convenient road. For example, in the case of a river with low banks and much frequented by shipping, the construction of a bridge which would at the same time afford an uninterrupted passage for vessels under it, and a convenient means of transit for vehicles over it, would be almost impossible; because, were the arches of the bridge made of sufficient height to allow vessels freely to pass through them, the level of the roadway would be so much elevated above that of the adjoining banks as to require either a very steep approach from both sides, or long and expensive embankments; and if, on the other hand, the level of the roadway were so low as to remove these objections, the passage of vessels with masts would be stopped, and the navigation materially interfered with.

To remove these difficulties in such a situation, it has been suggested to construct the bridge with one or two of its arches so arranged as to be capable of being opened, at intervals, for the passage of vessels. But such an arrangement only mitigates the evil, and is far from entirely removing it; and, in those situations where the traffic over the bridge was considerable and continuous, the periodical stoppages occasioned by the opening of the passages for the navigation would be extremely inconvenient.

The preservation of a free channel for the navigation, and the formation of a convenient means of communication, are, however, not the only important points to be considered. The effect which the construction of the bridge is likely to produce upon the river itself—whether it would tend to increase the velocity of the stream, and so produce a scour and washing away of the sides and bed of the river, and ultimately, perhaps, endangering its own existence; whether it would alter, and in what manner, and to what extent, the existing direction of the current, and so cause the formation of eddies and still water, and their constant attendants, shoals and banks of deposit; or whether the obstruction of its piers might not, in times of floods, by damming back the water, occasion the overflow of tracts of country adjoining the river above the site of the bridge—are all inquiries of immense importance, which require to be duly weighed and considered previous to determining the proportions which ought to be given to the several parts of the structure.

There are three different kinds of bridges, namely, those of masonry, those of iron or timber, and those on the suspension principle, each of which is peculiarly adapted for certain situations and circumstances; and, therefore, the *kind* of bridge to be adopted is also a point to be considered in the first stage of the investigation, although there are certain situations for which one kind of bridge would be as well suited as another, in which cases the choice becomes

merely one of taste, or is determined by pecuniary considerations.

With regard to the selection of the best site for a bridge, in many cases very little room is left for the exercise of the engineer's judgment in the matter, the position of the bridge being determined by other circumstances, such as the necessity of joining two existing roads, or of avoiding interference with existing establishments on the river's banks. Where, however, the choice of its position is left with the engineer, it becomes necessary for him to make a careful personal inspection of the locality, to have the banks of the river accurately surveyed, as well as soundings taken of the depth of the river at uniform distances apart, and borings of the nature of the strata composing its bed. In addition to which, he should inform himself of the velocity of the stream, the height of the water, and the quantity passing down the river at all times and seasons of the year, as well as the nature and extent of the trade carried on upon it. Prepared with these data, he will be in a position properly to consider the subject, and to arrive at correct conclusions. In the infinite number of different cases and varying circumstances which may arise in practice, it would be impossible to lay down any general laws upon this subject, but the following hints will be found of service in guiding to a correct determination. That part of the river should be selected whose course is straightest, bends and sharp turns being very unsuitable situations for a bridge, because at such parts the stream is usually irregular, and not parallel to the river's banks; and unless the piers of the bridge were placed parallel to the direction of the stream, which in such a situation they could hardly be, they would offer a much greater obstruction to the motion of the water, occasion eddies and shoals below the bridge, be liable to be undermined by the action of the stream acting only on one side of the pier, and endanger the safety of vessels navigating the river by their

tendency to be carried against the piers. For the same reasons, the bridge, if possible, should cross the river at right angles to the course of the stream, and, if this be prevented by circumstances, the piers must still be placed parallel to the stream, and making an angle with the direction of the bridge. An arch so constructed is termed a *skew* arch, and the angle formed by the sides of the piers and a line perpendicular to the direction of the bridge is termed the angle of skew, and should not exceed seventy degrees. Not only must the engineer, however, be careful to construct his piers in such a manner as not to alter the *direction* of the stream, he must also see that its *velocity* is not materially increased, which, if the bed of the river is of a soft or loose nature, would cause it to be rapidly scoured away, and, by undermining the foundations of the bridge, in time endanger its stability. The following table, taken from the *Edinburgh Encyclopædia*, exhibits the velocity of stream, which, under ordinary circumstances, the various descriptions of soil enumerated are capable of resisting.

The ordinary nature of current.	Velocity.		Materials that resist these velocities, and yield to more powerful ones.
	In feet per second.	In miles per hour.	
Very slow	0·25	0·171	Wet ground, mud.
Gliding	0·50	0·341	Soft clay.
Gentle	1·00	0·682	Sand.
Regular	2·00	1·364	Gravel.
Ordinary velocity	3·00	2·046	Stony.
Extraordinary and rapid floods	3·35	2·284	Broken stones, flints.
Extraordinary floods and rapids	3·45	2·352	Collected pebbles, soft schists.
	3·55	2·420	Beds of rocks.
Torrents and cataracts	9·86	6·723	Hardened rocks.

It should, however, be observed, that the scouring influence of rivers depends not only on the *velocity* with which they move, but also upon the *depth* or *weight* of water resting upon their bed. The reason of this will be readily understood if we consider that the water acts upon the materials

composing the bed of the river by its *friction* very much in the same manner as a solid body would; and it therefore follows that the more heavily the water is pressed against the ground, the greater will be its friction, and the more powerful its scouring agency; in addition to which, when the depth is considerable, the increased pressure of the water causes it the more readily to insinuate itself into the interstices of the strata, which it thus loosens and assists in breaking up. It is, however, only in the first act of breaking up the bed of the river that an increase of the depth of water influences and increases its scouring effect; the power of the water to carry or roll onward the matters which it has torn up depends only upon its velocity, always supposing that the quantity of the water is such, that its momentum is not influenced by the resistance of these matters, and that the depths are within such limits that the specific gravity of the water is not materially increased by compression.

The most important point claiming the attention of the engineer, as far as the stability of the bridge is concerned, is to obtain a secure and unyielding foundation for the piers and abutments, such as will safely support the superincumbent weight of the bridge and its load, and is not likely to be affected or disturbed by changes in the bed of the river, or other circumstances.

It may here be generally observed, that while bridges of masonry are best adapted for the support and conveyance of heavy and continuous traffic, such as that passing daily over London Bridge, they offer greater obstruction both to the stream and navigation than either iron or suspension bridges, on account of the *comparative* smallness in the span of their arches, and the piers occupying a space varying, in the examples which we have given in the tables following,* from $\frac{1}{3 \cdot 66}$th or $\frac{3}{11}$ths of the span, as in the bridge of the Holy Trinity at Florence, to $\frac{1}{9 \cdot 26}$th or $\frac{4}{37}$ths, as in the Neuilly

* See Tables, pp. 226 and 230.

Bridge over the Seine; while those of the iron bridges vary from $\frac{1}{7 \cdot 8}$th or $\frac{5}{39}$ths, as in Vauxhall Bridge, to $\frac{1}{14 \cdot 4}$th or $\frac{5}{72}$nds, as in the Pont du Carrousel; and those of the suspension bridges from $\frac{1}{8 \cdot 8}$th or $\frac{5}{44}$ths, as in the Brighton Chain Pier, to $\frac{1}{43 \cdot 5}$th or $\frac{2}{87}$ths, as in the bridge near Fribourg. These numbers may be taken as those representing the comparative obstruction which the respective bridges offer to the stream; but that which affects the navigation depends not only upon the width or span, but also on the height or headway afforded under the arch of the bridge; so that we should, as far as the facilities which each offers to the navigation, rather compare the area of the space between the intrados of the arch and the surface of the water; and this we have done in the following table, taking as an example of bridges of masonry, London Bridge; of iron, Southwark Bridge; and of suspension bridges, that across the Thames, near Charing Cross. There is another point, in respect of which the suspension bridges and those of iron are preferable to those of masonry, and this is their much smaller weight, a point which we have illustrated by a comparison of theweight of the centre arches and piers of the three bridges above mentioned. Although, however, with these advantages, they may be also considered as much less costly, there

Name of bridge.	Distance between centres of piers.	Span of centre arch.	Thickness of the piers, the span of the arch being unity.	Weight upon base of centre pier.		Average weight of the superstructure, for 1 foot in length and 1 foot in width.	Thickness from soffit to roadway, at the crown.	Area of clear space between the intrados of the arch and high water; the greatest headway multiplied by the span being unity.
				Of the pier itself.	Of the superstructure.			
	Feet.	Feet.		Tons.	Tons.	Tons.	Ft. In.	
London Bridge.........	176	152	·158	6830	14615	1·800	8 2	·7854
Southwark Bridge ...	264	240	·100	6580	3080	·314	8 0	·7173
Charing Cross Bridge	676½	646	·047	5140	535	·061	1 9	·9340

are certain situations in which undoubted preference should be given to bridges of masonry. Such are those in which the traffic is continuous and heavy, or the site much exposed to hurricanes and tempests; in either of which case a suspension bridge would not be advisable, as would not an iron bridge in the former.

CHAPTER XVII.

BRIDGES OF MASONRY.

Having already stated, at some length, the principles upon which the equilibrium of arches of masonry depend, and given rules for finding the pressure upon the keystone by which its depth should be determined, and, that being fixed for so proportioning the depth of the other parts of the arch that the whole may be in equilibrium, it only remains here to offer a few remarks upon the practical use of these rules. By reference to the table on page 230 it will be seen how widely the practice of engineers differs with regard to the load which they consider it safe to place upon an arch of masonry. Taking into consideration the materials of which it is composed, the bridge which carries the Great Western Railway across the Thames, near Maidenhead, is certainly the boldest which has ever been constructed, the actual pressure at the crown of the arch being about one-third of that which would begin to injure the cohesive strength of the material of which it is composed. And, although the construction of this bridge has shown that it is practicable to approach much closer to the load which would cause failure than had before been considered safe, it is questionable how far prudence would warrant such an approach in ordinary cases, especially when we consider how many accidental circumstances may deteriorate the stability of the arch, to guard against which it seems desirable that a much wider margin should usually be given, and that the

greatest load upon the key-stone should not be greater than $\frac{1}{20}$th of that which would *begin* to crush its material in bridges exposed to only ordinary traffic, and in those which are continually exposed to the tremour and vibration occasioned by a continuous and very heavy traffic not more than $\frac{1}{30}$th.

We have in the annexed table exhibited the proportions and dimensions of some of the principal bridges in Europe, in which we have given the radius of curvature of the main arch at its soffit, as well as the depth of their key-stones and the materials of which they are composed, from which the student will be enabled to observe the proportions which have been adopted by some of the most eminent engineers.

TABLE OF THE DIMENSIONS OF SOME OF THE PRINCIPAL BRIDGES OF MASONRY.

Nos. of reference to Fig. 140.	Name and situation of bridge.	Number of arches.	Clear waterway.	Total length, including piers.	Width of bridge between parapets.		Form of the centre arch.	Dimensions of Centre Arch.					Material.	Date of completion.	Architect or Engineer.
								Span.	Rise or versine.	Radius of curvature at the crown.	Depth of the key-stone.	Thickness of the piers.			
			Feet.	Feet.	Ft.	In.		Feet.	Feet.	Feet.	Feet.	Feet.			
1	Vielle-Brioude, over the [illegible]	1	183½	. .	16	0	Circular.	183·25	70·25	94·87	5·25	. .	Tufa	1454	Grenier and Estone.
2	[illegible]	3	270½	322½	33	9	Slightly Pointed	95·25	14·83	172·63	2·75	26	White Marble	1569	B. Ammannati.
3	[illegible]	1	140	. .	. .		Circular	140	35	87·5	2·5	. .	Sandstone	1755	Edwards.
4	[illegible]	3	358	409	35	5	False Ellipse	127·83	38·25	134·6	6·25	25·58	Saillancourt Stone	1765	Perronet and Kupeau.
5	[illegible]	9	780	926	42	0	Idem	100	41·5	56	6·58	20	Portland Stone	1771	Mylne.
	Neuilly Bridge, over the Seine	5	639	766	48	0	Idem	127·83	31·83	260	5·25	13·83	Saillancourt Stone	1774	Perronet.
6	Bridge of St. Maxence, over the Oise	3	230	249	41	6	Circular	76·67	6·25	121	4·67	9·5	. .	1784	Idem.
7	Waterloo Bridge, over the Thames	9	1080	1240	41	6	Ellipse	120	32	112·5	5·0	20	Granite	1816	Rennie.
8	[illegible] lge, over [illegible]	1	150	. .	35	0	Idem	150	35	160	4·5	. .	Sandstone	1827	Telford.
9	[illegible], over the [illegible]	5	692	784	53	6	Idem	152	29·5	162	4·75	24	Granite	1831	Rennie.
10	[illegible] Dee, at Chester	1	200	. .	33	0	Circular	200	42	140	4·0	. .	Sandstone	1833	Hartley.
11	Bridge carrying the Great Western Railway across the Thames, at Maidenhead	2	256	284	28	0	Ellipse slightly Pointed	128	24·25	169	5·25	23	Bricks laid in Cement	1835	Brunel.

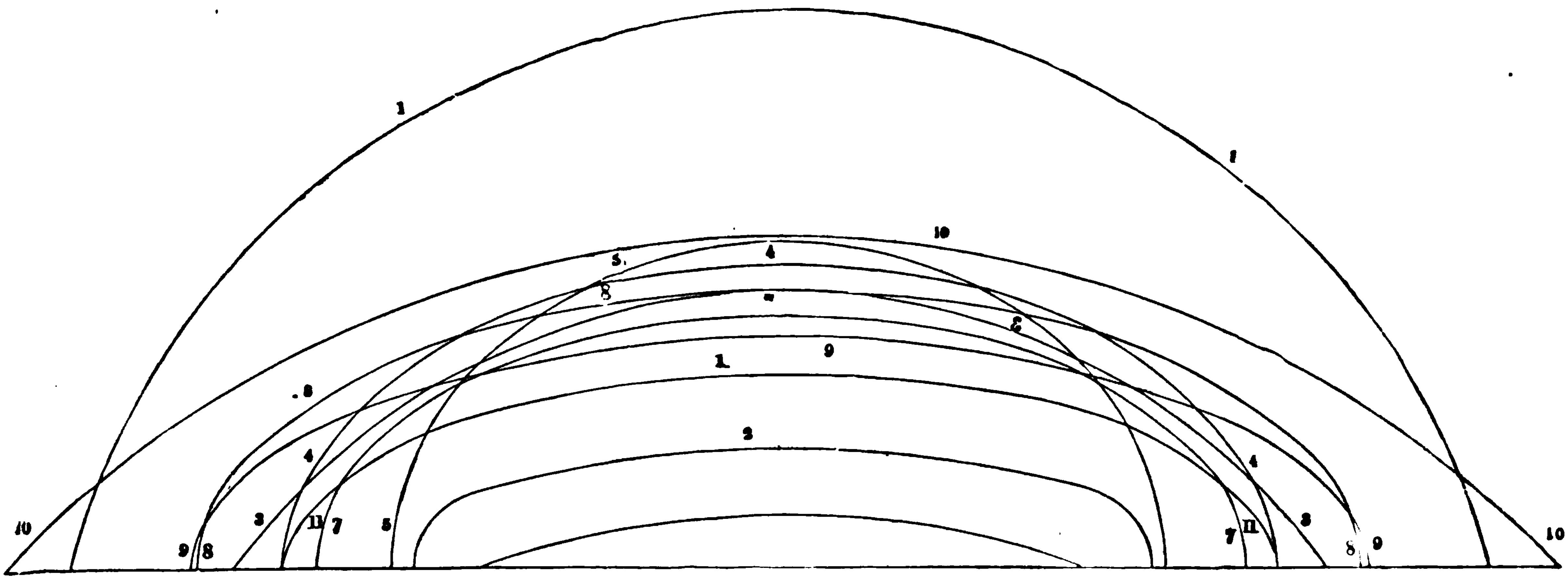


Fig. 140.—Arches of Stone Bridges.

In the accompanying figure, 140, we have shown the intradoses or profiles of the principal arch of each of the bridges mentioned in the foregoing table, all drawn to the same scale, so as to afford at one view a comparison of their relative size and form.

In the construction of a bridge the most important point is to obtain an unyielding foundation for the piers and abutments, and, if this can be secured, the engineer may with safety adopt bold proportions for the arches of his bridge; but, in a situation in which the piers would be likely to settle to any extent, every precaution should be taken to increase the stability of the arches. It is a matter which may reasonably excite surprise that engineers should so universally construct the piers of their bridges with solid masonry, since a very little consideration would suffice to show that such a mode of construction is usually the worst which could be adopted, especially where the ground beneath the piers is of a yielding nature. The real office which the pier of an arch is intended to perform is merely to support the arch, to receive its weight, and transmit it to the foundation, and it performs this in the most perfect manner when it adds to that weight in the least degree; in most cases, however, the weight of the pier itself is equal to about half that of the superincumbent arch,* so that the weight which the foundations have to carry is half as much again as the real weight of the bridge. In the construction of the piers of a bridge, the points which ought to be attended to are as follows, namely, that the substance of the pier shall be sufficient to enable it to sustain without injury the vertical pressure of the arch and its load, as well as that of the water and any

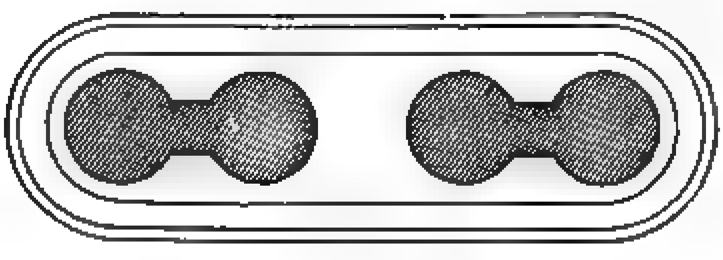
* By reference to the table at page 226, it will be seen that the weight of that portion of the pier of London Bridge *below* the springing is nearly equal to half that of the centre arch; and that, in the case of Southwark Bridge, the weight of the pier is more than twice as great as that of the superstructure, which it merely serves to support.

accidental force to which it might, under extraordinary circumstances, be liable to be exposed; and that its base should be of such dimensions, that the pressure arising from its own weight, and that which is insistent upon it, may be distributed over a sufficiently large area of ground. Now, so long as these two conditions are fulfilled, it is sufficient; and any additional substance given to the pier is clearly so much additional load thrown upon the foundations, and is positively detrimental to the stability and security of the structure.

Perronet appears to have understood better the real use of piers, although he seems to have been more disposed to lighten them by reducing their external dimensions rather than by constructing them hollow; for instance, in the Neuilly Bridge, already mentioned, we find the piers are less than one-ninth of the span of the arch. In the bridge of St. Maxence he has, however, effected the same object by carrying up the piers in four columns united in pairs, and turning a small arch across between them intersecting the main arch of the bridge, as shown in Figs. 141.

The foregoing remarks apply with equal force in the case of abutments as in that of piers, the usual practice in the construction of which has been to form a solid mass of masonry, the weight of which materially *assists* the thrust of the arch in producing settlement by the compression of the ground upon which it rests. It is obvious that the real use of an abutment to an arch is nothing more than to extend the surface upon which it rests and from which it derives support, without at the same time materially increasing its pressure, and so, by reducing the load on any given area, to increase the stability of the structure; whereas it would be found, in the majority of cases, that the pressure upon every square foot of the surface at the springing *is less than* that which the thrust of the arch and the additional weight of the abutment together occasion on the foundation upon which they rest.

In building a structure, the weight of which is considerable, upon any kind of substratum (excepting only rock), some amount of sinking or settlement from the compression of the ground will almost always be found to take place, and as it is very desirable, in the case of a bridge, that the settlement of the piers and abutments, if any, should take place previous to the construction of the arch, the piers and abutments, when built up to the springing course, should be



Figs. 141.—Bridge of St. Maxence.

loaded with a weight at least equal to that of the arch which they are afterwards to carry; and in this state they should be left, if possible, for some months, during which period the water should be admitted into the interior of the coffer dams, so that the piers may be brought as nearly as possible into the same condition as that in which they would be when the bridge was completed; so that, if the ground is disposed to yield under the joint influence of the water and the load,

it may do so before the construction of the arches is commenced.

Previous to the piers being loaded, and at regular intervals afterwards, careful levels should be taken to ascertain whether any settlement has occurred; and as soon as it has been found by means of these observations that all subsidence has ceased, and not until then, the arches should be commenced. The loading of the piers should be gradually removed as the arches progress, in such a manner that the weight upon the piers may be maintained as nearly uniform as possible.

Next in importance to securing a firm foundation for the piers and abutments is the proper construction of the arch itself. Could the arch-stones or voussoirs be worked with perfect accuracy to the wedge form required, and then be brought into immediate contact without the interposition of any mortar or cement, as was frequently done by the Romans and the Cyclopæan builders of old, we should have an arch in the highest perfection, not liable to settlement, and which would maintain its form unaltered as long as the materials of the stone endured. Although, however, we cannot in practice dispense entirely with mortar between the joints, they may be reduced so much in thickness as to leave but small room for any after-settlement in the arch arising from their compression; and, by proper attention to these points, engineers have so far succeeded as to be able to construct arches of two hundred feet span, with a settlement in the crown of the arch of scarcely 2½ inches.

To support the voussoirs of the arch during its construction and until the insertion of the key-stone, it is requisite to have a timber platform termed the *centre* or *centering*, the upper surface of which is made to correspond accurately with the intrados of the arch, so that the stones being placed upon it may be retained in their proper position, until the arch is completed by the insertion of the key-stone.

It is requisite that the centre of an arch, of any size, should be constructed with the greatest possible care, and in such a manner that the weight of the arch-stones may not alter its form; a point very difficult to be secured with a material so elastic as timber, and where the load is at first thrown only on a small portion of the framing. In cases where this has not been sufficiently attended to, it has been found requisite to place a load upon the middle of the centres, to counteract their tendency to rise at that point, occasioned by the depression of their haunches under the weight of the arch-stones. And in the centres for the Neuilly Bridge, designed by Perronet, from their peculiar mode of construction, the settlement was so considerable (nearly two feet at the crown) that a variety of expedients had to be resorted to, to prevent their being crushed, and letting down the arches. It is also requisite in the centre of an arch to have the means of gradually lowering the centre as soon after the completion of the arch as may be deemed prudent, which should be done in the most regular and gradual manner, in order that the arch may have time to settle equally; this operation is technically termed *striking* the centres, because they are usually supported on wedges, the striking out of which allows of its gradual descent, in the manner which we have described.

As an illustration of the practical operations involved in the construction of a stone bridge, we have selected the Grosvenor Bridge over the Dee, at Chester, not only on account of the boldness of the arch, but also because several novel expedients were adopted by its engineer, Mr. Hartley, of Liverpool, with very great success. The profile of the arch is shown by the line 10 in Fig. 140, and the principal dimensions of the bridge will be found in the table at page 230. Fig. 142 is an elevation of the bridge, and Fig. 143 a longitudinal section of half the arch and the north abutment, showing the centre upon which it was constructed.

The south abutment of the arch is founded upon the solid rock, as is also the principal portion of that on the north side; but the rock suddenly terminating at A, and being succeeded by a stratum of very loose sand, it was found necessary to drive piles for the support of the back portion of the abutment, as shown in the figure. The material of which the bridge is constructed is the native sandstone, with the exception of the face of the abutments and the two first courses of the arch, which are of granite, and the three centre courses of the arch and the quoins, which are of Anglesea marble or limestone. By an inspection of the plate it will be seen that the principle of the arch is carried out in the abutments, the courses of which are made to radiate towards the centre of the intrados of the bridge, until they meet the rock, in which steps were cut, the bed of which partook of the same slope, so that the rock itself may be regarded as the actual abutment of the arch.

Upon striking the centres of bridges, it is usually found that, in consequence of the compression in a greater or less degree of the mortar in the joints of the voussoirs, the form of the arch becomes modified, on account of the greater settlement of the stones in the centre or crown of the arch. The

Fig. 142.—Grosvenor Bridge, Chester.

reason of which is, that as the stones approach the haunches they become less inclined to the horizon, and a greater portion of their weight is thrown upon the joint, less being borne by the centre, from which cause the compression of the joints near the haunches takes place during the construction of the bridge; whereas, in those stones which are near the crown of the arch, their weight being almost entirely borne by the centres, the joints are but slightly compressed until the weight of the stones is brought upon them by the operation of striking the centres, and then the settlement consequent upon this compression takes place. We have already explained, while treating of the stability of arches, that, when the crown of an arch sinks, the tendency of the arch-stones near the crown is to turn upon their outer edges, and of those near the haunches upon their inner edges, in the manner shown in Fig. 136, the effect of which is frequently seen in the opening of the joints at the back of the arch at the haunches, and on the soffit of the arch near the crown, pieces being frequently splintered off from the opposite edges of the joints in consequence of this tendency to turn about them.

Now in the Chester Bridge this tendency of the joints to open was guarded against by the insertion of thin plates of lead between the arch-stones on each side, from the springing up as far as that point in the arch where the line of pressure passes through the centre of the stones, which in this case was assumed to be at about one-third of the arch; and further, by two wedges of lead being laid under the springing course, which were an inch and a half in thickness on the face of the arch, and ran out to nothing at the back. By these means, as the arch settled, the lead, being of a yielding nature, became slightly compressed, and caused the pressure to be more equally distributed over the surface of the joints. The following method was also adopted of setting the key-stones, by which the joints near the crown

B

A

Fig. 143.—Centering for Grosvenor Bridge.

of the arch were somewhat compressed previous to the centres being struck: three thin strips of lead were placed on the sides of each of the stones composing the last course on each side of the key-stones, which latter, having been besmeared with a thin kind of putty, composed of white lead and oil, were forced down into their places by a small pile-driving engine, the strips of lead serving as slides to prevent the stones rubbing against each other.

We have yet to describe the centre, which was designed by Mr. Trubshaw, the contractor for the bridge, and differed very materially from any which had been previously constructed. It was supported upon four temporary piers of stone, built in the river, two of which are shown in the section (Fig. 148). From the tops of these piers the timbers which were intended to support the arch-stones radiated in the manner shown in the drawing, their lower end being secured in a cast-iron shoe fixed on the pier for their reception, and their upper ends being connected together and retained at nearly equal distances apart by two thicknesses of planks bent round to the form of the arch; and they were still further secured by the horizontal timbers to which they were bolted. There were six of each of these fan-like framings in the width of the bridge, placed at equal distances apart, and steadied by transverse timbers. The timbers for the support of the arch-stones, technically called *laggings* (one of which was placed under every joint), were supported upon the curved rim of each of the framings, folding wedges being placed under them, so that, by driving the wedges back, any portion of the arch might be gradually lowered at pleasure. The peculiarity in the construction of this centre consisted principally in the timbers being disposed radially, so as to receive the pressure of the voussoirs in the direction of their length, after the manner of a pillar, in which direction timber, when subjected even to very considerable strains, suffers very

slight compression; and these centres were not, therefore, liable to the failing of too many others, that of change of form, under the unequally distributed load of the arch while in course of construction. The manner in which the centres were struck was also peculiar, that of having separated wedges under each arch-stone, so that any portion of the arch might be relieved from support, while the remainder was still borne by the centre; and thus the engineer possessed the power of allowing those parts of the arch to settle first which he might think desirable. Whereas in the ordinary form of centre it is usual to have the entire span of the arch in one framing supported upon wedges at each extremity, upon striking which the whole of the centre would be lowered simultaneously.*

The method which we have above described, of inserting strips of lead between the joints of the voussoirs, was adopted with the same object in the construction of a bridge over the Dora Riparia, near Turin. In this instance the engineer constructed the centre with a greater rise than that which he intended the arch to have when completed, so as to allow for its settlement: the span of the arch was 147·64 feet, and the versine or rise 18·04 feet, while that of the centres was made equal to 18·9, or about 10 inches greater. The arch-stones, which were of granite, having been accurately formed to the proper wedge-form, were then put in their places on the centre, in such a manner that the joints near the haunches were made wider on the face of the arch than at the back, those midway being made parallel, and those near the crown wider at the back than on the face of the arch, no mortar or cement being placed between the stones, which were kept at the proper distances apart by wedges of iron and lead driven in between them. When the whole arch had thus

* For a further description of this bridge, and plates showing the details of its construction, the reader is referred to the *Transactions of the Institution of Civil Engineers*, vol. i. p. 207.

been completed, and the position of the arch-stones carefully examined, a moderately liquid cement composed of equal portions of lime and clean sand was poured into the joints. After which, being allowed twenty days to consolidate, the centres were gradually struck, when the arch subsided with great regularity about $4\frac{3}{4}$ inches, and a load of about 8,000 tons of ballast being uniformly distributed over the arch, and allowed to remain for four months, caused a further settlement of $1\frac{3}{4}$ inch, but without producing any irregularity in the form of the arch.*

The elevation (Fig. 144) of the bridge constructed by Telford over the Severn, at Gloucester, has been introduced for

Fig. 144.—Bridge over the Severn, at Gloucester.

the purpose of pointing out a peculiarity in the form of its soffit, first suggested by Perronet, which consists in making the curve of the intrados of the arch flatter at the face than in the middle of the arch, so as to form a kind of splay on each side, commencing at the haunches and dying away at the crown where the two curves are made to coincide, and at which point alone the soffit of the arch is straight on the transverse section. In the example which we have selected, the form of the arch in the centre is an ellipse, as shown by the line 8, Fig. 140, while the line of the intrados on each of the external faces of the bridge forms a flat segment of a

* For a full account of this work, see the *Transactions of the Institution of Civil Engineers*, vol. i. p. 183.

circle. Perronet himself applied this peculiar mode of forming the soffit in the Neuilly Bridge, over the Seine, already referred to, the dimensions of which have been given in the table at page 230. The same principle was also adopted in the bridge which we have mentioned above as being constructed over the Dora Riparia, near Turin; but in this case both the curves are segments, only the external one is much flatter than the other. In addition to the pleasing effect of lightness and grace which this method of forming the soffit of an arch affords, it possesses some advantage in saving of material, as well as affording a better form (somewhat resembling that of the contracted vein) for the passage of water, in cases where the river, in time of floods, is liable to rise above the springing of the arch.

CHAPTER XVIII.

CAST-IRON BRIDGES.

THE principle which has usually been adopted in the construction of bridges of cast iron is to support the roadway upon separate ribs, each of which partakes of the properties of an arch, being subjected in like manner entirely to a *compressive* force. They differ, however, essentially from an arch of masonry, in respect of the parts of which these ribs are composed (and which answer to the voussoirs of the arch) being so securely connected together as to prevent the possibility of rotation about their edges, should the line of pressure deviate beyond the substance of the rib. In an arch of masonry, the object is so to proportion the depth of the arch in every part that it may be equilibriated, or, in other words, that the line of pressure may everywhere pass directly through the centre of every one of the joints of the voussoirs. In an arched rib of cast iron, on the contrary, the object is so to form the framing of the ribs and spandrils (which, although in separate parts, should be so connected together as to be one) as to insure the utmost rigidity and stiffness combined with lightness. It is, then, a matter of small importance, whether the line of resistance passes exactly along the centre of the rib, because the whole semi-arch may be looked upon as one huge voussoir, supported at its lower end upon the pier, and at its upper extremity by the equal and similar pressure of the other semi-arch.

We have already given a rule by which the crushing strain

on the rib at the crown of the arch may be determined; it may, however, be desirable to illustrate its practical application, for which purpose we have selected Southwark Bridge. In this case, the weight of half the centre arch, with the roadway, is about 1,520 tons, and the horizontal distance of the centre of gravity of the same from the springing about 56 feet,* and the versine or rise of the arch is 24 feet. Then we have from the rule, as 56 is to 24, so is the horizontal thrust to 1,520, which gives 3,547 tons for the horizontal thrust at the crown of the arch, or the strain tending to crush the cast-iron ribs. This strain may be supposed, without any sensible practical error, to be equally borne by all the ribs, of which there are eight; and the sectional area of each being about 214 square inches, we have for the compressive strain upon every square inch of the ribs about 4,650 lbs., or only $\frac{1}{23}$rd of that which would be required to crush the material.†

In the following table we have collected the principal dimensions of a few of the more important cast-iron bridges

* The distance of the centre of gravity of the whole mass, from the springing of the arch, is found in the manner already explained, by multiplying the weight of each separate part by the distance of its centre of gravity from the springing, and dividing the sum of the products thus obtained for all the parts of the bridge by the weight of the whole mass.

† It has been supposed by some that in Southwark and many other iron bridges, little or no additional strength is derived from the arched form of the ribs, and that the real strain to which they are exposed is similar to that of a girder supported at each end and loaded with a distributed weight, there being scarcely any horionztal thrust. That such is not, however, the case, is sufficiently evident by comparing the weight which girders of the same dimensions as the ribs of the bridge, and in the circumstances supposed, would be able to support, with the load which they actually sustain; for, by the rule given at page 12, we find that the weight which would *break* such a girder would be 108 tons, or about 870 tons for the eight ribs of Southwark Bridge, which is less than a fourth of that which they have sustained for many years.

which have been constructed. And in Figs. 145 we have shown the sectional forms which have been adopted in each case for the main ribs. We have already mentioned that in girders exposed to a transverse strain, their strength may be materially increased by adopting a particular form of cross section; in the case, however, of the ribs of a cast-iron bridge, where they are entirely exposed to a compressive strain, the form of the cross section is immaterial, always, however, supposing that the rib is sufficiently stiff to prevent any tendency to bend laterally or sideways. The form of those (shown at g in the subjoined figures) of the bridge over the Lary appears to be the best adapted for this purpose, the side webs or flanges imparting considerable lateral stiffness to the ribs.

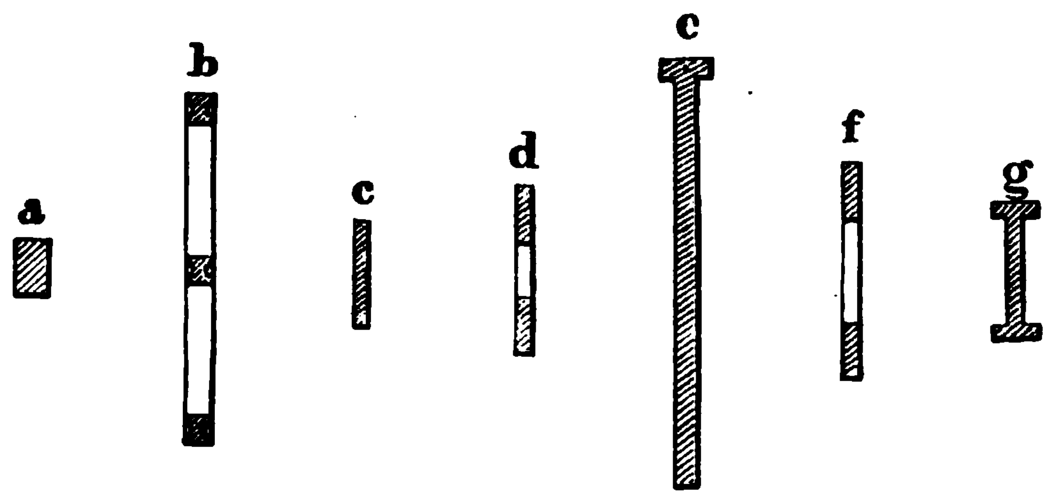


Fig. 145.—Sections of Ribs of Cast-iron Bridges.

TABLE OF THE DIMENSIONS OF SOME OF THE PRINCIPAL BRIDGES OF CAST IRON.

Reference to Fig. 145.	Name and situation of bridge.	No. of arches.	Width of bridge between parapets.	Form of the centre arch.	Dimensions of centre arch. Span.	Rise or versine.	No. of ribs.	Vertical depth of ribs.	Area of each rib.	Thickness of the piers.	Manner in which the roadway is supported.	Date of completion.	Engineer.
			Ft.		Feet.	Feet.		Feet.	Sq. ins.	Feet.			
a	Colebrook Dale, over the Severn	1	26	Circular.	100·5	45·0	5	0·75	56·25	—	Cast-iron Plates.	1779	Darby.
b	Sunderland, over the Wear	1	32	Idem.	240·0	30·0	6	5·00	46.50	—	Platform of Timber.	1796	Wilson.
c	Buildwas, over the Severn	1	18	Idem.	130·0	17·0 34·0	3 2	1·25 1·50	37·50 47·25	—	Cast-iron Plates.	1796	Telford.
d	Over the Avon, at Bristol.........	1	31	Idem.	100·0	15·0	6	2·33	32·00	—	Idem.	1806	Jessop.
f	At Bonar, over an arm of the Sea..................................	1	16	Idem.	150·0	20·0	4	3·00	45·00	—	Idem.	1812	Telford.
g	Vauxhall Bridge, over the Thames	9	36	Idem.	78·0	14·0	10	—	—	10	Idem.	1816	Walker.
e	Southwark Bridge, over the Thames	3	42	Idem.	240·0	24·0	8	6·00	214·00	24	Idem.	1818	Rennie.
f	Tewkesbury Bridge, over the Severn	1	24	Idem.	170·0	17·0	2 4	3·00 3·00	45·00 36·00	—	Idem.	1826	Telford.
g	Over the Lary, near Plymouth	5	24	Segment of Ellipse	100 0	14·5	5	2·00	64·00	10	Idem.	1827	Rendel.
—	Pont du Carrousel, over the Seine................................	3	35	Idem.	187·0	15·5	5	3·33	165·00	13	Platform of Timber.	1836	Polonceau.

[Two cast-iron arched bridges, Figs. 146 and 146*a*, were designed by Mr. John Fowler for the Coalbrookdale Railway and the Severn Valley Railway, crossing the river Severn. The arch, of cast-iron, has a span of 200 feet, and a rise of 20 feet; it consists of four ribs, as in Fig. 146, 4 feet deep, flanged at the upper and lower edges to a width of 15¼ inches; and of 2 inch metal throughout. The horizontal, upper members or

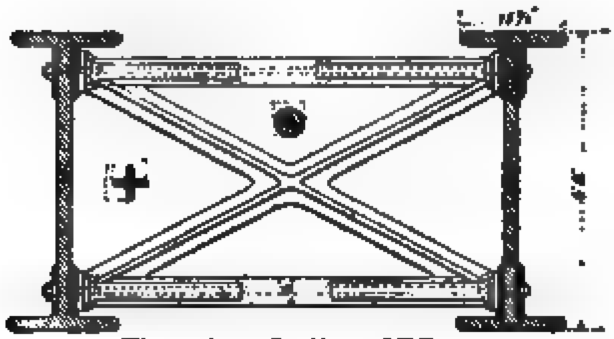


Fig. 146 *a*.—Section of Ribs.

girders, are of wrought-iron plate, 2 feet deep, 12 inches wide at the upper and lower flanges, of ½-inch plate. Each arch was cast in nine segments, bolted together, and braced laterally, and the stress is limited to 2½ tons per square inch of section. The girders are closely connected to the arches, at the crown and in the spandrils.]

CHAPTER XIX.

WROUGHT-IRON BRIDGES.

[WROUGHT-iron arched bridges have been constructed on lines of railway. The most recently erected bridge of this class, Fig. 147, is the railway bridge over the river Tyne, at Wylam, Northumberland, on the line of the Scotswood, Newburn, and Wylam Railway. The bridge is constructed with two lines of rails. It is of one span of 240 feet—of the form of a "free arch," with a suspended roadway. The arch is formed of three wrought-iron lattice ribs, springing from a level 19½ feet below that of the rails, and having a clear rise of 48 feet. The platform is supported on 19 cross girders dividing it into 20 bays of 12 feet each; most of the girders are suspended from the arch. The roadway consists of four continuous plate girders of 250 feet each, one under each rail, which rest on and are riveted to the cross girders. The rails are laid on longitudinal waybeams, resting on the top of the roadway girders, which are prolonged 25 feet beyond them, and are bolted to cross sleepers where they clear the masonry. By this combination the platform is steadied, and any endway movement other than that due to expansion is checked. The depth of the ribs is 10 feet at the heel and 7 feet at the centre. In each rib the upper and the lower members, or booms, are what may be called semi-cellular, consisting of two sides and a horizontal plate, connected by angle-irons at the corners. The sides of each boom are of channel iron, 9¼ inches by 4 inches, $\frac{9}{16}$ inch

Fig. 147.—Wylam Bridge.

thick. The lattice bars are of T iron. The suspension bars are attached to a pair of angle-irons riveted to the lower booms of the ribs. They are, in section, 9 inches by $\frac{3}{4}$ inch for the centre rib; and $7\frac{1}{2}$ inches by $\frac{9}{16}$ inch for the outer ribs. The foundations are laid on beds of cement concrete; they consist of a double course of ashlar covered by granite impost stones, having a bulk of 60 cubic feet each. The weight of materials in the superstructure is as follows:—

	Tons.	Tons.
Cast Iron	3	
Wrought Iron	279	
		282
Timber (2,160 cubic feet)		54
Total weight		336

The cost of the bridge, including every charge, was:—

		Per square foot of platform, viz., net span by net width.
	lbs.	s. d.
Abutments	5,500	15 6
Superstructure	10,500	29 0
Total cost	16,000	44 6

The greatest deflection of the bridge, when tested, took place under the weight of three locomotives and tenders,

weighing together 166 tons, on one line of rails—1·08 inches at the outer rib, and 0·48 inch at the centre rib. There was no permanent set produced, though the maximum load placed on the bridge—one line of rails—amounted to 333 tons, when the deflection of the outer rib was 0·72inch, and that of the central rib 0·36 inch. When both lines were loaded, under a total of 333 tons, the deflection of the outer ribs was 0·96 inch, and that of the central rib, 1·20 inch. The general results of the testing experiments were—a maximum deflection of $\frac{1}{2400}$ of the span under the most unfavourable conditions of loading; no perceptible lateral vibration, either of the ribs or of the platform, under a heavy moving load; a perfect recovery of form in the ribs after a heavy load, both stationary and passing, with no permanent set; a very slight "road-wave" in front of an advancing train, barely $\frac{1}{8}$ inch.

The loads adopted for the calculations were:—

	Centre Rib. Tons per foot.	Side Ribs. Tons per foot.
Structural load	0·7	0·4
Moving load	1·3	0·7
Total	2·0	1·1

The strains (stresses) allowed for the various parts were—

Flanges of the ribs	4	tons per square inch.
Struts of the latticing	3	" "
Suspension bars	4½	" "
Platform girders	4	" "

The only other bridge of this kind is the one erected by Mr. Leather in 1833, for a carriage road over the river Aire, at Leeds, to a span of about 140 feet. The ribs of this bridge are of cast iron, and they are not braced internally. But they support a suspended roadway.*

* See Mr. W. G. Law's paper on the "Railway Bridge over the river Tyne, at Wylam, Northumberland," *Proceedings of the Institution of Civil Engineers*, vol. lvi. page 262.

Girder bridges of wrought iron have been brought extensively into use for railways as well as for common roadways. The early forms for such girders are typified in Fig. 148, showing in section one of the wrought-iron girders of the Torksey Bridge, erected in 1850. The bridge is constructed of two spans of 130 feet each. The girders are parallel in elevation, and are 10 feet deep. The upper boom is cellular, of plates $\frac{3}{8}$ inch and $\frac{5}{16}$ inch thick at the middle of the span; the two sides, enclosing a hollow space, are of $\frac{1}{4}$-inch plates; and the lower boom is formed of two or three plates riveted together, two of them $\frac{5}{8}$ inch thick and the third $\frac{3}{4}$ inch thick. The limiting stress was 5 tons per square inch of section.

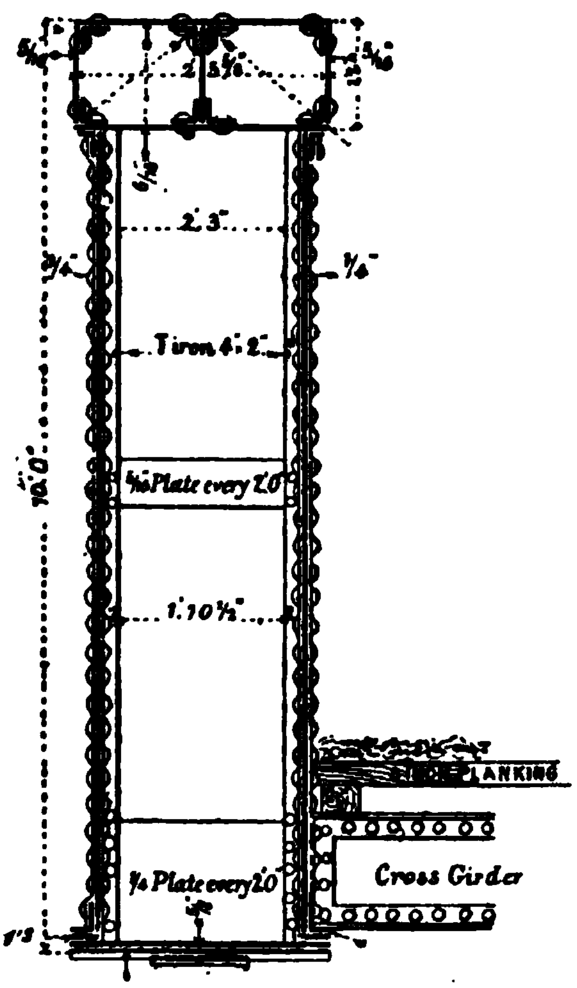


Fig. 148.—Girder Bridge.—Section.

Lattice-girders are now almost universally adopted for iron bridges, combining lightness, strength, and economy in construction. Of English desig, two bridges may be mentioned exhibiting extremes of practice in the design and proportion of lattice-girder bridges. The first is the iron lattice bridge forming a portion of the viaduct across the Boyne River, on the line of the Dublin and Belfast Junction Railway, near Drogheda. It has three large openings, of which the middle space is 264 feet, and the side spaces are 138 feet 8 inches. The latticing, which is connected over the piers, forms one continuous beam. The beam or girder is parallel, $26\frac{1}{2}$ feet deep; the lattice bars, each of which is crossed by six others at the angle 45°, form squares. The platform of the bridge is $24\frac{1}{2}$ feet wide, to

carry two lines of rails, and rests on cross lattice-girders fixed to the main girders. The top and bottom booms or tables of the main girders are each formed of a pile of plates 3 feet in width, to a combined thickness of $2\frac{1}{8}$ inches above, affording a sectional area of $113\frac{1}{4}$ square inches at the middle of the girder, and $3\frac{1}{2}$ feet by $2\frac{3}{4}$ inches thick below, with a sectional area of 127 square inches. In the central span the weight of wrought iron amounts to 386 tons, and of cast iron to 5 tons. The maximum calculated stress does not exceed 5 tons per square inch for compression, and $4\frac{1}{2}$ tons per square inch of nett section for tension. The maximum load on the bridge was taken as 2 tons per lineal foot.*

The other lattice bridge is the railway bridge over the Thames at Charing Cross, of which Sir John Hawkshaw was the engineer. This bridge comprises nine spans: six spans of 154 feet, and three spans of 100 feet. Its total length is 1,375 feet. It was constructed for four lines of rails, and is formed with a fan-like expansion at the Charing Cross end, where it terminates at the station. The width of the river at the bridge when it was constructed was 1,350 feet. The bridge was built on the site of the old Hungerford Suspension Bridge, the two brick piers of which have been retained and utilised for the present bridge. The greatest depth of water between the two brick piers is 13 feet below low-water spring tides, and the average depth is about 10 feet. The rise of spring tides is $17\frac{1}{2}$ feet. The level of the rails is 31 feet above Trinity high-water mark, and the clear minimum headway is 25 feet above the same level.

The piers for the spans of 154 feet, other than the brick piers, are cast-iron cylinders, 10 feet in diameter above ground, and expanded to a diameter of 14 feet in the ground. Each pier consists of two such cylinders, Fig 149,

* See Mr. J. Barton's paper on Wrought-iron Beams in the *Proceedings of the Institution of Civil Engineers*, vol. xiv. page 443.

placed at a distance apart of 49 feet 4 inches between centres. The piers are constructed of segmental plates, generally 1⅛ inch in thickness, with flanges and 1¼-inch bolts and nuts. The lowermost ring of plates is 1¼ inch thick, except at the lower edge, where the thickness is increased to 1¾ inch. The Act for the railway provided that the foundations of the bridge should be laid at such a depth as would admit of the river being deepened, at any subsequent time, to

CROSS SECTION OF OPENINGS 154-FEET SPAN

Fig. 149.—Charing Cross Bridge.

30 feet below Trinity high-water mark. The cylinders between the two brick piers were sunk to a depth of 62 feet below this level. The piers next the Surrey side were sunk to a depth of 52 feet below the same level. They were sunk by excavating the material from the inside, by means of divers with helmets, until they had passed through porous materials into the London clay. The weight with which it was found necessary to load the cylinders, in order to overcome the friction of the

sides, and to sink them to their final depth, averaged about 150 tons. The London clay at the base of the cylinders was of a hard character, of a dark-blue colour; it extended to a depth far below the level at which the cylinders were founded. After the cylinders had been sunk and the material had been excavated, the wider base of the cylinders was filled with concrete, and thence with brickwork to the underside of the granite-bearing blocks or caps on the cylinders. The concrete was composed of Thames gravel, or ballast, and Portland cement, in the proportion of 7 to 1. The brickwork is of best pavior bricks, set in Portland cement mortar, which is mixed in the proportion of 1 of cement to 2½ of sand. Under a load of 700 tons, the cylinders thus filled sunk further to a permanent depth of 4 inches. The granite bearing blocks are 2½ feet thick, in two semicircular halves, to fill the cylinders, and standing 1 inch above the upper edges of the cylinders; so that the load may not bear directly upon the cylinders, but should be taken entirely by the filling. Each pair of cylinders forming a pier is connected transversely by a wrought-iron box girder, 4 feet deep. Four lines of way are carried on the bridge, and if they were loaded with locomotive engines, the pressure on the base of the cylinders would amount to about 8 tons per square foot if no deduction be made for the frictional resistance of the sides of the cylinders, or about 7 tons if such allowance be made.

In the superstructure of each opening of 154 feet span there are two main girders, which are supported on the piers, and are, like the cylinders, 49 feet 4 inches apart from centre to centre transversely, carrying between them four lines of rails. The way is supported on cross girders, which are fixed to the main girder. These girders also overhang the main girders, to carry the footpath at each side. The main girders are of wrought iron, 14 feet deep, placed independently from pier to pier. The upper and lower

booms are each formed of two sides and a horizontal member of plate iron riveted together with angle iron, so proportioned as to support, under a maximum load, 4 tons per square inch compressive stress in the upper boom, with a sectional area at the centre of 300 square inches; and 5 tons tensile stress in the lower boom, with a solid sectional area of 235 square inches, after deducting the sectional area of the rivet-holes. The sides of the girders are constructed in panels, divided by vertical bars, each panel containing two diagonal bars, crossed at the angle 45°, pinned to the booms by steel pins 7 inches in diameter at the ends of the girders and 5 inches at the centre. The upper boom is composed of five plates, at the centre $\frac{5}{8}$ inch thick, 4 feet broad; and the lower boom, of five $\frac{5}{8}$-inch plates and one $1\frac{1}{8}$-inch plate, 3 feet wide, drilled for and united by 1-inch rivets, spaced at a pitch of 4 inches. The weight of one main girder is 190 tons.

The total weight of metal work in the bridge is:

Wrought-iron work and steel pins	. . . .	4,950 tons.
Cast-iron work		1,950 „
	Total . .	6,900 „

The superficial area of the roadway and the footpaths together amount to 103,000 square feet. The total cost of the bridge, including the abutments, was £18,000, being at the rate of £1 15s. per superficial foot, or £131 per lineal foot.*

The Fink truss bridge, designed by Mr. Albert Fink, represents a class of iron bridge extensively employed in the United States. In this bridge a pair of diagonal tension-bars connects the foot of the central strut with the ends of the upper boom. Each half-span is similarly subdivided into two quarters, and each quarter into eighths, and

* See Mr. Harrison Hayter's paper on the Charing Cross Railway Bridge, *Proceedings of the Institution of Civil Engineers*, vol. xxii. p. 512.

each eighth is subdivided by a shorter strut. A Fink bridge was erected in three spaces of 205 feet each on the line of the Baltimore and Ohio Railroad, across the Monongahela River. The two trusses are 16 feet apart from centre to centre for a single line. The tension-rods are attached to the main struts at points 22 feet 8 inches below the centre of the upper booms, or about $\frac{1}{9}$th of the span. The upper booms and the central strut are of cast iron, octagonal in section, hollow, and 12 inches diameter across the sides; they are 10 inches in diameter externally, and present a sectional area of 41 square inches. The main tension-rods in each truss are each formed of six bars, $4\frac{1}{2}$ inches by $1\frac{1}{4}$ inches, making a sectional area of $33\frac{3}{4}$ square inches. The other vertical struts are of octagonal cast iron, 8 inches in diameter externally and 7 inches internally. This bridge, for a single line, weighs, with permanent way included, only $\frac{1}{2}$ ton per lineal foot. With an additional load of 1 ton per lineal foot, the tensile stress on the wrought-iron ties is calculated not to exceed 5·15 tons per square inch, and the compressive stress on the cast-iron $4\frac{1}{4}$ tons. The way is carried near the level of the bottom of the truss on a timber platform, which is steadied by distance-pieces inserted between the lower ends of the struts. The cost of the Fink truss bridge, thus executed, was £14 per lineal foot, single line, against from £5 to £7 per lineal foot for trussed timber bridges.

Employment of Steel for the Construction of Bridges.

The use of steel presents advantages in comparison with iron for the construction of bridges, combining lightness with strength. With the limiting stress, $6\frac{1}{2}$ tons per square inch of section, authorised by the Board of Trade, the material of a steel bridge may be 23 per cent. less than that required for

an iron bridge, for which the limiting stress is 5 tons per square inch. Mr. James Price, who has investigated the question, has concluded that, with respect to large swing-bridges, in the design of which lightness is a specially desirable consideration, a sixth of the weight might properly be economised in the substitution of steel for iron.]

CHAPTER XX.

SUSPENSION BRIDGES.

EQUILIBRIUM OF SUSPENSION BRIDGES.

IN a suspension bridge, the roadway or platform is suspended from chains, the links of which are straight, by vertical rods attached to the joints. And as the chains are not rigid, but are capable of altering their form by motion about any of the joints, it follows that, in any position which the chain assumes, its several parts must be in equilibrium. Now a chain so circumstanced in no way differs from a polygonal framing, such as is shown in Fig. 131, supposed to be inverted, excepting that the strains upon the several bars or links, which in the latter were thrusts tending to compress the bars, are now tensile strains tending to pull them asunder; and therefore all the properties of the one are common to the other, and the various relations which we have shown to subsist between the weights suspended from the angles, the strains on the several bars or links, and the horizontal strain in the case of the polygonal framing, similarly subsist in that of the chains of a suspension bridge. The investigation, however, necessary for deducing from these relations rules for determining the proportions of a suspension bridge, that its several parts may be in equilibrium, involves the use of propositions and terms in mathematics far too abstruse and difficult to be admitted in this place; we must, therefore, content ourselves with pointing out the circumstances affect-

ing their stability, and merely giving rules for proportioning their parts, without attempting their demonstration.

The chains of a suspension bridge have to support three separate loads, which are very differently distributed, namely, their own weight, which varies with the dimensions of the chain and its inclination; the weight of the rods by which the chains and platform are connected, and which varies with their length; and the weight of the platform or roadway with its load, which is usually uniformly distributed. The first suspension bridges which were constructed had their chains made of the same dimensions throughout; but as the tensile or pulling strain upon the different parts of the chain varies greatly, depending, in fact, upon its inclination, being greatest at the points where the chains are attached to the piers, and least in the centre or lowest point of the chain, it is evident that in so constructing them a superabundance of strength is given to the centre portion of the chain, and that the strength of the whole would be increased by taking away some of the metal from those parts of the chain and adding it to the parts more inclined, so proportioning their substance that the cross section of the chain may be in every part proportional to the strain which that part has to sustain.

Let Fig. 150 represent a suspension bridge, with the roadway or platform F L horizontal, and A D B C being the curve formed by the chains; the points A and C, at which the chain is attached to the piers, are called the *points of suspension:* the horizontal distance A E or E C of these points from the centre of the bridge, the *semi-span;* and the vertical distance E B of the lowest point of the chain below the point of suspension is termed the *deflection.* The term *sectional area of the chains,* at any point, means the surface (measured in square inches) which would be exposed by sawing the chains across at that point.

The first point to be determined in the case of a suspension

bridge is the form of the curve A D B C which the chains will assume, and upon which will depend all the principal dimensions of the bridge. The dimensions which are requisite for

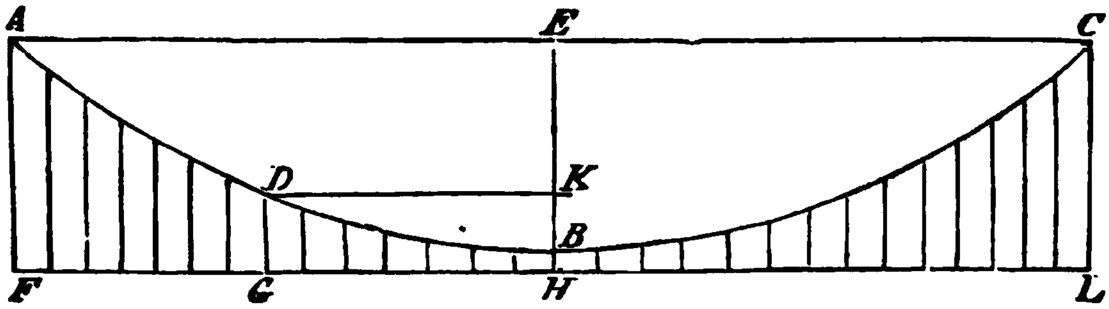


Fig. 150.

determining this are the semi-span A E, the deflection E B, and the distance B H, of the roadway below the lowest point of the chain, or the length of the shortest suspending rod; these being known, any number of points in the curve may be determined by the following rule:—*

THE ROADWAY OF A SUSPENSION BRIDGE BEING HORIZONTAL, TO FIND THE LENGTH OF THE SUSPENDING ROD D G AT ANY POINT D.—Subtract the length of the shortest suspension rod B H from the deflection E B; multiply the remainder by the square of the horizontal distance D K of the point D from the lowest point B of the chain, and divide by the square of the semi-span A E; to the quotient add the length of the shortest rod B H, and it will give the length of the suspending rod D G.

The curve formed by the chain having been found, it only remains to determine the strains to which each portion of it is exposed, in order that its area in every part may be made proportional to the strain which that part has to sustain. In order to determine these, it is necessary to have, in addition

* These rules are deduced from the formulæ given by Professor Moseley in his "Mechanical Principles of Engineering," in which work he has given a very able and complete investigation upon this difficult subject. In the above rules the tensile strain required to break a square inch of wrought iron is taken at 67,200 pounds, the weight of a bar a foot long and an inch square, at 3·3 pounds, and the iron is supposed to be loaded with only a sixth of its breaking weight.

to the dimensions above, the weight of a foot in length of the roadway or platform of the bridge, including the greatest load which it is ever possible that it will have to support. These being known, the following rules will give the dimensions of the chains.

TO FIND THE STRAIN ON THE LOWEST POINT B OF THE CHAIN, AND ITS SECTIONAL AREA.—Subtract the length of the shortest suspension rod B H from the deflection E B; divide twice the remainder by the square of the semi-span A E, and from the quotient subtract ·0008; divide the weight in pounds of a foot in length of the roadway when loaded by this remainder, and the quotient will be the strain in pounds upon the lowest point B of the chains; and if this strain be multiplied by ·0000893 it will give the sectional area of the chains in square inches at the same point.

TO FIND THE STRAIN ON THE CHAIN, AND ALSO ITS SECTIONAL AREA AT ANY POINT D.—Divide twice the vertical height K B of the point D above the lowest point B of the chain by the horizontal distance D K of D from B, and to the square of the quotient add 1; the square root of this sum multiplied by the strain on the chain at B (as found by the rule above) will give the strain upon it at D; and this strain multiplied by ·0000893 will give the sectional area of the chain at the same point in square inches.

We shall illustrate the use of these rules by an example. Let the semi-span be 200 feet, the deflection 40 feet, the length of the shortest suspending rod 2 feet, the weight of a foot in length of the roadway when loaded 5,000 lbs., and the horizontal distance D K of the point D from the centre of the chain 100 feet.

Then, by the first rule given above, 2 subtracted from 40 leaves 38, which multiplied by the square of 100 equals 380,000, and this number divided by the square of 200

gives as the quotient 9½, to which, adding 2 feet, the sum 11½ feet is the length of the suspending rod D G.

By the second rule, 2 subtracted from 40 leaves 38, twice this number divided by the square of 200 equals ·0019, from which, subtracting ·0003, the remainder equals ·0016; then 5000 divided by this number gives 3,125,000 lbs. for the strain upon the lowest point B of the chain; and 3,125,000 multiplied by ·0000893 equals 279 square inches for the sectional area of the chain at B.

And by the third rule, twice 9·5 divided by 100 équals ·19, the square of which added to 1 equals 1·0261; then the square root of 1·0261 equals 1·013, which multiplied by 3,125,000 gives 3,165,625 lbs. for the strain upon the chains at the point D; and 3,165,625 multiplied by ·0000893 gives 283 square inches for the sectional area of the chain at the point D.

In the case of bridges of masonry and iron, both from the weight of the structures themselves as well as from the rigid nature of the material, their forms are not liable to be altered or their equilibrium disturbed by external influences, such as those arising from the wind or the transit of heavy loads. With suspension bridges, however, the circumstances are very different, and it has been found that they are materially influenced by these external forces, and in some cases have sustained very serious injuries from them. The reason of this is to be found, not only in the extreme lightness of the superstructure of such bridges, in consequence of which but a very slight force is required to put them in motion, but also from their peculiar susceptibility to vibration, or undulatory motion, arising from the centre of gravity of the structure being *below* instead of *above* the point of support, and from the chains being in a state of tension, somewhat similar to the strings of a musical instrument, so that the sudden application of a considerable force to any part of the chain, or the continued and regular impulse of even a slight

force, would cause the chains to alter their form, and throw both themselves and the platform into a state of vibration. Thus, suppose the whole line A B D, in Fig. 151, to represent the position of one of the chains of a suspension bridge while in its natural state, and then let us suppose a weight to be suddenly brought upon any point E of the platform, about half-way between the points of suspension and the centre of the bridge. Now the effect which this weight will produce will be that of depressing the platform below its ordinary level, and also drawing down the chain by means of the suspension rods, and causing it to assume the form shown by the lower dotted lines ; the depression of the chain at F will, however, be attended by an elevation at G, on the opposite

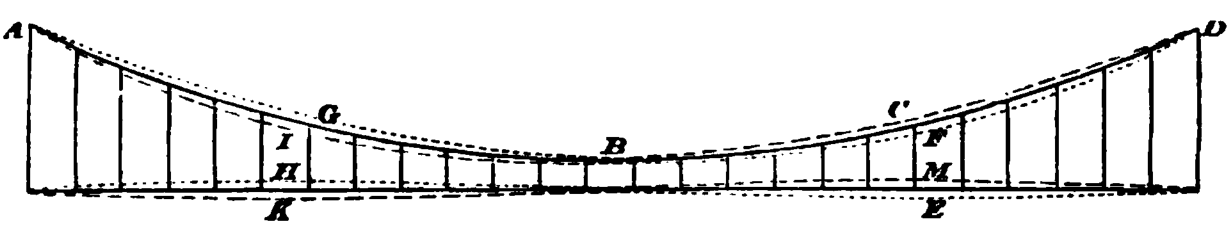


Fig. 151.

side of the centre of the bridge, and a corresponding elevation in the platform. The form of the chain will, therefore, now become as shown by the dotted line A G B F D, and the platform, instead of being level, will have assumed the waved or undulatory form shown by the dotted line H E. If, now, this weight be again suddenly removed, the chain and platform will immediately return to their former positions ; but in doing so they will have acquired a certain velocity and momentum, sufficient to carry them as much beyond their proper position in the opposite direction, and the chain and platform will assume the form shown by the dotted lines A I B C D and K M, in which the parts previously depressed are now elevated, and *vice versâ* ; this position will, however, be only momentary, and they will once more return nearly to the position which they at first assumed when under the influence of the weight. And thus they will continue in a

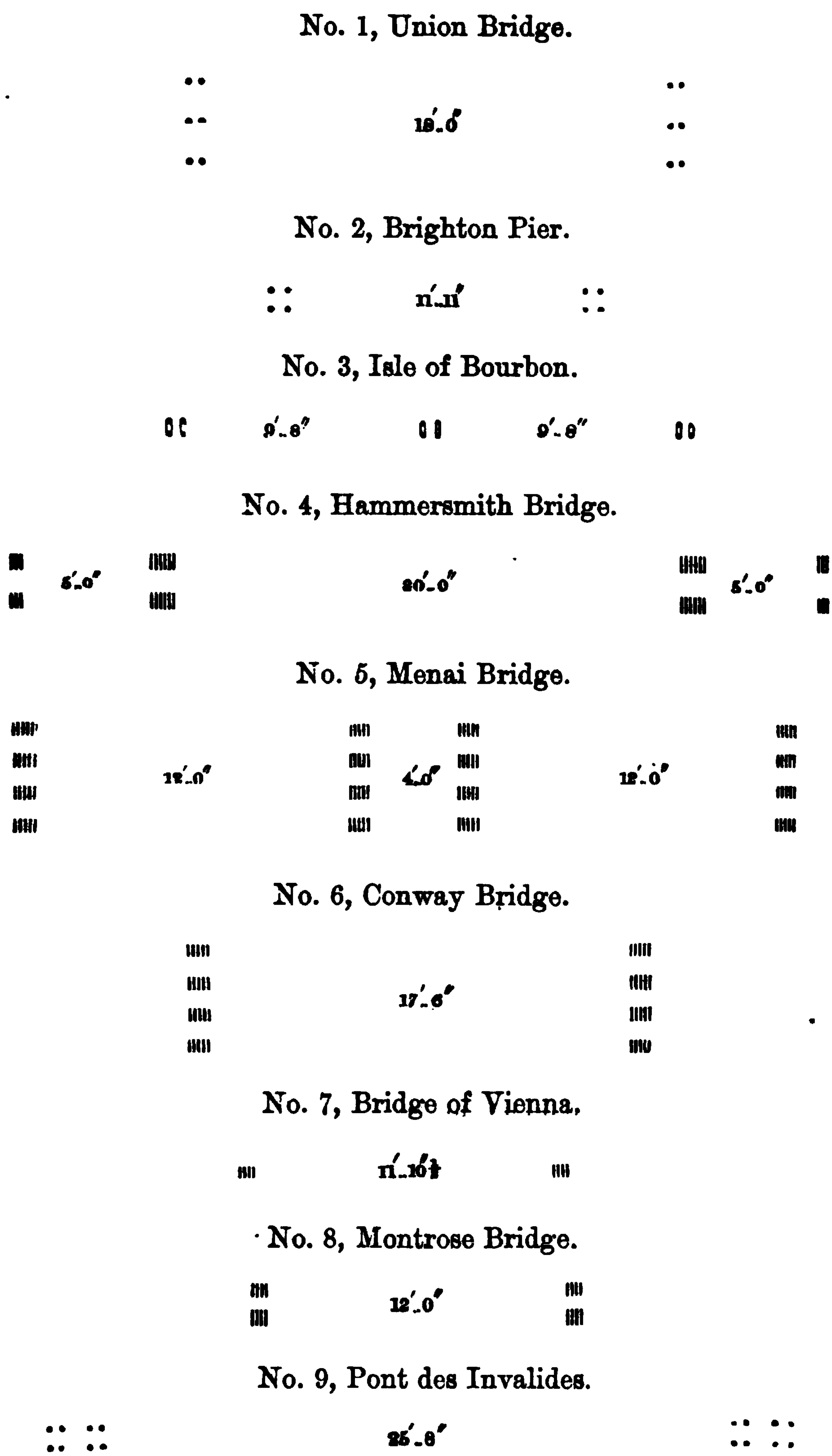
state of vibration until the effects of the disturbing force has been gradually absorbed by the resistance of the chains and platform to motion.

In all cases it is important to render the platform itself as stiff and rigid as possible, and, further, to connect the chains on each side of the bridge so together as to constitute essentially but one chain, as in those of the Charing Cross Bridge, so that, their weight being greater, they will require a more considerable force to put them into motion than where the chains are separate, as in the Menai Bridge.

We have in the following table given the chief dimensions of some of the principal suspension bridges which have been constructed, either in this country or abroad; and in Figs. 152 we have given transverse sections of the chains, showing the arrangement and disposition of the links composing them which has in each case been adopted.

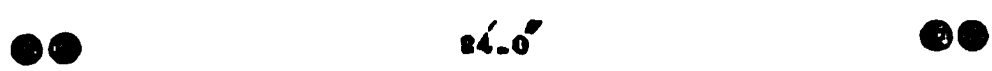
TABLE OF THE DIMENSIONS OF SOME OF THE PRINCIPAL SUSPENSION BRIDGES.

Nos. of reference to Figs. 152	Name and situation of bridge.	Span of the catenary formed by the chains.	Deflection of the catenary formed by chains.	Deflection in parts of the span = unity.	Breadth of the platform of the bridge.	Thickness of pier at the level of the platform	No. of separate chains.	Sectional area of the chains in the centre of the bridge.	Size of the separate links composing the chains.	Date of completion.	Engineer.
		Feet.	Feet.		Ft. in.	Feet.		Sq. in.			
1	[illegible] ed	449	80	·067	18 0	17·5	6	38	2 ins. in diameter.	1820	Sir Samuel Brown.
2	[illegible]	255	18	·071	13 0	29	4	25 13	Idem.	1823	Idem.
3	[illegible] n	220·3	25·49	·116	20 0	12	8	17 4	1·36 ins. in diameter.	1823	Sir I. Brunel.
4	[illegible] mes	422·25	29·5	·070	30 0	22	8	180	5 ins. × 1 in.	1824	Tierney Clarke.
5	[illegible] ts	570	43	·075	28 0	29	16	260	$3\frac{1}{4}$ ins. × 1 in.	1826	Telford.
6	[illegible] Conway, over an arm of the sea	327	22·33	·068	17 6	—	8	130	Idem	1826	Idem.
7	Bridge over the Danube at Vienna	334	21·4	·065	11 $10\frac{1}{2}$	21·4	2	15·6	Steel bars, 2·42 ins. × 0·8 in.	1828	Herr von Mitis.
8	Montrose Bridge over the Este	432	42	·097	12 0	20	4	80	5 ins. × 1 in.	1829	Sir Samuel Brown.
9	Pont des Invalides, over the Seine	236·5	26·33	·111	25 8	15·5	8	41·6	1·61 ins. in diameter.	1829	M. Navier.
10	Fribourg Bridge, across the valley of the Sarine	870	63	·072	21 3	20	4		Chains composed of 4,224 separate wires, each 0·12 in. in diameter.	1834	M. Chaley.
11	Charing Cross Bridge, over the Thames	676·5	50	·074	14 0	30·5	4	296	7 ins. × 1 in.	1845	Isambard K. Brunel.

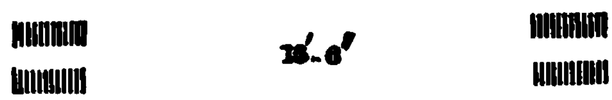


Figs. 152.—Suspension Bridges—Sections of Chains.

No. 10, Fribourg Bridge.



No. 11, Charing Cross Bridge (late).



Figs. 152 (*continued*).

The chains of the Union Bridge, No. 1, the Pier at Brighton, No. 2, the Bridge in the Isle of Bourbon, No. 3, and the Pont des Invalides, No. 9, are formed of rods of round iron; the others, with the exception of the bridge over the Danube, No. 7, and the Fribourg Bridge, No. 10, are formed with flat bars of wrought iron, grouped together in chains, in the manner shown in the figures. The chains of the bridge over the Danube are of steel, a material adopted by the engineer, H. Mitis, on account of its great strength combined with lightness. It is, however, very questionable whether this supposed advantage is not the reverse, since from the extreme lightness of the chains of this bridge, as compared with the weight of the platform (the latter being nearly five times as heavy as the former), the bridge is found to vibrate considerably under the influence of heavy loads or high winds, notwithstanding the extreme flatness of the curve formed by its chains, the deflection being less as compared with the span than that of any of the other bridges mentioned in the table. The chains of the Fribourg Bridge are composed of an assemblage of wrought-iron wires, formed into a bundle or cable, but not twisted; each cable is composed of twelve strands containing each fifty-six wires, and eight strands containing each forty-eight wires, making in the total 1,056 wires in each chain or cable. The use of wire as a material for the chains of suspension bridges has been very general on the Continent, and, in many respects, it is well adapted for the purpose. It has, however, been urged

against its use, and with some reason, that it is peculiarly liable to corrosion, the fabrication of the chain being favourable to the secretion and retention of moisture within the interstices between the wires by capillary attraction; and the danger of the interior wires being by these means corroded, without the possibility of its being detected by observation. In the case of the Fribourg Bridge, this evil was guarded against by immersing each wire, three several times, for two hours, in a mixture of boiling linseed oil with a small quantity of litharge and soot; and the same composition was afterwards *payed* over the separate strands and the finished cables.

With regard to the arrangement of the chains, that adopted in the Menai and Conway Bridges, Nos. 5 and 6, namely, of having four separate chains, and placing them vertically over each other, is not good, in consequence of the large surface which they thus present to the wind, and, being separate, the slight force required to throw them into motion. This disadvantage was very evident in the case of the Menai Bridge during the storm of January 7, 1839, when the lateral motion of the chains was so considerable, that, although suspended at a distance of 12 feet apart (as shown in the section), they had, after the breaking of the transverse ties and tubes, been thrown so violently against each other as to cause deep indentations in the iron and to break off the heads of the bolts, the shanks of which were 3 inches in diameter.

In arranging the proportions to be given to the several parts of a suspension bridge, the spans and deflections of the contiguous openings must be so adjusted, that the horizontal strains produced by the chains on each side of the pier shall be equal, and consequently balance each other; for otherwise, unless the saddle to which the chains are connected were fixed, it would be drawn off the pier in the direction of the greater strain, and, if it were fixed, the stability of the

pier would be endangered from the tendency of the greater strain to pull it over. It may easily be ascertained whether this equality in the horizontal strains exists or not, in the following manner: having assumed certain proportions for the two openings, calculate, by means of the rule already given, the strain upon the chains of each opening (taking as the point D that in which the chains meet the pier); the strains thus obtained will be those acting in the direction of the chains, and, in order to ascertain the equivalent horizontal strains, we must, by means of the rule already given, find two points in each of the chains near the pier, from which we shall ascertain their directions, and we may then easily find the amount of the horizontal strains by resolving each of the strains acting in the direction of the chains into two others, one acting vertically and the other horizontally, in the same manner as has been already explained. Should it thus be found that the horizontal strain produced by the chains on one side of the pier would be greater than that produced upon the other, their relative proportions must be varied until they are made to balance each other.

Clifton Suspension Bridge.

[The late Hungerford Suspension Bridge, across the Thames at Charing Cross, was removed to make way for the erection of the railway bridge already described. The materials of the bridge were utilised in the construction of the Clifton Bridge across the Avon at Bristol, for which Mr. W. H. Barlow and Sir John Hawkshaw were the engineers. The bridge has a span of 702 feet 3 inches, and the versed sine of the curve of the chains is 70 feet, or about one-tenth of the span. The width of the bridge, including roadway and footways, is 31 feet, the chains are 20 feet

apart between centres, and the roadway is curved upward with a rise of 2 feet. The height of the roadway above high-water level is 248 feet.

The chains are carried upon the piers by wrought-iron saddles, placed on roller frames of cast iron, the rollers being of cast steel. The beds of the roller frames are laid at an angle of 1 in 20, rising towards the river. At a distance of 196 feet from the centres of the piers landwards, land saddles are placed, without rollers, bedded upon brickwork of Staffordshire blue bricks, in cement set upon the solid rock. At an additional distance of 60 feet, at an inclination of 45°, the chains are carried to the anchorage. In this length they diverge to a distance of 5 feet apart, and they are inserted through the castings which form the anchorage plates—one plate to each chain. Each anchorage plate, 5 feet by 6 feet, is bedded upon a mass of brickwork set in cement, built in the form of an arch in plan, abutting upon the solid rock.

The sectional area of the chains at the piers is 481 square inches, and at the centre of the span 440 square inches. The weight of the chains between the piers is 554 tons. The stress on the chains at the centre of the bridge caused by the weight of the chains themselves is nearly 680 tons. The weight of the suspension rods, longitudinal girders, transverse girders, cross-bracing, hand-railing, roadway, &c., is about 440 tons, causing a stress, approximately, of 597 tons at the centre of the chains. The maximum moving load, estimated at 70 lbs. per square foot, amounts to 600 tons, which would cause a stress of 817 tons at the centre of the chain.

The total maximum stress at the centre of the chain is, then, as follows:—

Stress due to the chains	680
Ditto due to the weight of the platform, rods, &c.	597
Ditto due to the maximum moving load . .	817
Total maximum stress . .	2,094

Distributed on a sectional area of 440 square inches, the

maximum stress would amount to 4¾ tons per square inch of section of the chains. The stress due to the weight of the bridge itself is at the rate of 2·90 tons per square inch.

The suspension rods are each a little more than 2 square inches in section, on which the maximum stress would amount to 4¼ tons per square inch.

The maximum pressure on the brickwork cannot exceed 10 tons per square foot.

In order to provide for the effects of expansion and contraction, and to allow for the movement occasioned by wind, or by the passage of heavy loads across the bridge, the two extremities of the roadway are fitted with jointed ends or flaps, 8 feet long, which admit of perfect freedom of movement both vertically and in the direction of the length of the bridge.

The bridge was tested by a load of 500 tons of stone distributed over the surface. The total deflection under the load was 7 inches at the centre of the bridge, arising partly from the altered position of the saddles upon the piers. The cost of the bridge amounted to £34,975.

In gales of winds there is a horizontal deflection of the bridge, just perceptible. Secondly, an undulation from end to end—a slow and stately movement of the structure, a rising and falling of the roadway about halfway between the centre and the abutments, to the extent of about 6 inches above and 6 inches below the mean level. Thirdly, deflection of the land chains.*]

* See Mr. W. H. Barlow's paper on the Clifton Suspension Bridges in the *Proceedings of the Institution of Civil Engineers*, vol. xxvi., p. 243.

CHAPTER XXI.

MOVABLE BRIDGES.

[MR. JAMES PRICE, in the paper already referred to, classified Movable Bridges into—

1. Bascules.
2. Swings.
3. Traversing.
4. Vertical Lifts.
5. Pontoons.

From this paper the annexed Table of Movable Bridges is copied.

TABLE OF MOVABLE BRIDGES.

Locality.	Water Crossed.	Number of Passages.	Width of each Passage.	Nature of Way over.	Width of Bridge over all.	Number of Parts.	Total Weight of Moving Parts.	Motive Power.
			Ft. Ins.		Ft. Ins.		Tons.	
				BASCULES.				
Selby . . .	River Ouse	1	45 0	Railway	24 1	Two flaps	..	Manual.
Copenhagen . .	..	1	56 8	Highway	35 0	" "	205	Hydraulic compressed air, and manual.
Birkenhead . .	Great Float passage at granaries	1	30 0	Railway	12 0	" "	..	Hydraulic.
Dordrecht . .	Canal	1	42 0	Highway	..	" "	..	..
Drumsna, Ireland .	River Shannon	1	30 0	Railway	16 3	One flap	..	Manual.

Swings (A).

Hull . . .	Victoria Docks	1	45 0	Highway	24 0	Two leaves	..	Manual.
Brest . . .	Penfeld River	1	350 0	„	23 0	„ „	700	„
Liverpool . .	Waterloo Docks, entrances	1	60 3		27 0	„ „	..	Hydraulic.
Goole . . .	River Ouse	2	100 0	Railway	30 0	One leaf	670	„
Birkenhead . .	Entrance from Alfred Dock to East Float		30 0 and 50 0	„	42 0	„ „	..	„
Birkenhead . .	Entrance to Alfred Dock	1	100 0	„	42 0	„ „	450	„
Hull, South Bridge	River Hull	1	100 0	Highway	24 0	„ „	800	Hydraulic (originally manual).
Birkenhead . .	Passage from East to West Floats (Duke Street)	1	100 0	Railway	42 0	„ „	700	Hydraulic.
New Ross, Ireland .	River Barrow	2	50 0	Highway	32 0	„ „	159	Manual.
Londonderry . .	River Foyle	2	48 0	Railway below, highway above.	26 0	„ „	340	„

TABLE OF MOVABLE BRIDGES (*Continued*).

Locality.	Water Crossed.	Number of Passages.	Width of each Passage.	Nature of Way over.	Width of Bridge over all.	Number of Parts.	Total Weight of Moving Parts.	Motive Power.
			Ft. Ins.		Ft. Ins.		Tons.	
				SWINGS (B).				
Amboy, New Jersey	River Raritan	2	216 0	Railway	17 6	One leaf	501	Steam and hydraulic.
Atchison . .	River Missouri	2	160 0	"	..	" "	..	Steam.
Louisiana, Missouri	River Mississippi	2	180 0	"	16 0	" "	391	"
Keokuck, Iowa .	"	2	160 0	Highway and railway	20 0	" "	403	"
Burlington, Iowa .	"	2	160 0	Railway	15 0	" "	285	"
Quincey, Illinois .	"	2	160 0	"	15 0	" "	285	"
Hannibal, Missouri	"	2	160 0	"	18 0	" "	285	"
Rock Island . .	"	2	160 0	Railway, and highway below	20 0	" "	701	"

Dubuque . .	"	2	160 0	Railway	15 0	" "	285	"
Kansas City, Kansas	River Missouri	2	160 0	Railway and highway	20 0	" "	303	"
Buffalo, New York	River Niagara	2	160 0	Railway	18 0	" "	290	"
Albany, New York	River Hudson	2	112 0	"	28 0	" "	377	"
Providence . .	River Providence	2	104 0	Highway	44 0	" "	..	"
Athlone . .	River Shannon	2	43 0	Roadway	29 0	" "	130	Manual.
Galway . . .	Lough Athalia	2	60 0	Railway	38 0	" "	200	"
Hull . . .	River Hull	1	56 7	Highway	32 6	" "	240	"
Rochester . .	River Medway	1	48 6	"	40 0	" "	300	"
Cette . . .	Canal de Cette	2	65 7 and 46 0	Highway and railway	26 3	" "	220	"
Cette	Canal Maritime	1	69 0	Highway	13 1	Two leaves	84	"
Toulon . . .	La passe Missiessy	2	92 0 and 46 0	Railway and highway	22 8	One leaf	..	" "
Havre . . .	Great lock	1	100 0	Highway	22 0	Two leaves	..	"

TABLE OF MOVABLE BRIDGES (*Continued*).

Locality.	Water Crossed.	Number of Passages.	Width of each Passage.	Nature of Way over.	Width of Bridge over all.	Number of Parts.	Total Weight of Moving Parts.	Motive Power.
			Ft. Ins.		Ft. Ins.		Tons.	
			SWINGS B—(*continued*).					
Dunkerque . .	L'Ecluse de Barrage	1	69 0	Highway	13 3	Two leaves.	60	Manual.
Gravelines . .	L'Ecluse Vauban	2	33 0 and 26 3	„	13 3	One leaf	26¼	„
Stettin . . .	River Parnitz	2	40 0	Railway	24 6	„ „	100½	„
Dublin . . .	River Liffey	2	40 0	Highway	35 0	„ „	180	Manual or steam.
Grimsby . .	New Cut, Manchester, Sheffield, and Lincolnshire Railway	1	52 0	Railway	13 10	„ „	45	Hydraulic.
			SWINGS (C).					
Velsen, Holland .	North Sea Canal	2	70 0	Highway	15 7	One leaf	..	Manual.
Velsen . . .	„	2	68 0	Railway	20 0	„ „	228	„

SWINGS C—(*continued*).

Bergen-op-Zoom .	Kanaal Zindbeveland	2	52 0	„	22 4	„	„	216	„
Rotterdam . .	Dock passage	2	50 0	Highway	21 0	..		..	„

SWINGS (D).

Birkenhead . .	Morpeth, Dock and East Float	1	70 0	Railway	42 0	„	„	420	Hydraulic.
Liverpool . .	Canada Dock entrance	1	80 0	Highway	..	„	„	120	„
London . . .	Millwall Dock entrance	1	80 0	„	45 0	„	„	..	„
London . . .	Passage from Blackwall Basin to Export Dock	1	38 0	Double-line railway	27 4	„	„	400	„
London . . .	Passage from Blackwall Basin to Import Dock	1	37 3	„	27 4	„	„	325	„
London . . .	Passage from South West India Dock Basin to South Dock	1	55 0	Single-line railway	16 1	„	„	275	„
London . . .	Passage across east entrance to South West India Dock	1	55 0	„	16 6	„	„	175	„
Penarth . . .	Across lock	1	60 0	Roadway	13 6	„	„	170	Manual.

TABLE OF MOVABLE BRIDGES (*Continued*).

Locality.	Water Crossed.	Number of Passages.	Width of each Passage.	Nature of Way over.	Width of Bridge over all.	Number of Parts.	Total Weight of Moving Parts.	Motive Power.
			Ft. Ins.		Ft. Ins		Tons.	
			SWINGS D—(*continued*).					
Leith . . .	Water of Leith	1	120 0	Railway	39 0	One leaf	620	Hydraulic.
Marseilles . .	Entrance to Repairing Dock	1	91 10	Highway and railway	52 6	„ „	700	„
Hull . . .	Albert Dock entrance	1	80 0	Railway, single line, and highway	12 0	„ „	. .	Hydraulic.
Wisbeach . .	River Nene	1	85 0	Highway	33 0	„ „	500	Manual.
			SWINGS (E).					
Newcastle-on-Tyne	River Tyne	2	110 0	Highway	50 0	One leaf	Over 1,200	Hydraulic.
			SWINGS (F).					
Dublin . . .	Spencer Dock passage	1	28 0	Highway	30 0	One leaf	100	Manual.
Dublin . . .	Spencer Dock entrance	1	28 0	Highway and tramway for railway waggons	11 0	. .	. .	„

TRAVERSING BRIDGES.

Swansea . .	River Tawe, new cut	1	60 0	Railway	..	One piece	205	Hydraulic.
Swansea . .	Lock entrance	1	75 8	„	..	„ „	200	Manual.
London . . .	Millwall Dock entrance	1	80 0	Highway	32 0	One piece	200	Manual.
Hull . . .	River Hull	1	37 4	„	32 0	„ „	120	„
Birkenhead . .	Morpeth Dock	1	25 0	Railway	27 0	„ „	..	„
Dublin . . .	Spencer Dock entrance	1	26 0	Highway	30 0	„ „	..	„
South Wales . .	River Dovey	1	35 0	Railway	21 6	„ „	..	„
Lancashire . .	River Leven	1	36 0	„	..	„ „	..	„
Drumsna, Ireland .	River Shannon	1	30 0	„	16 3	„ „	45	„
Newry . . .	Canal	1	30 0	„	..	„ „	..	„
Greenock . .	Graving Dock	1	60 0	Railway and highway	16 3	„ „	204	„
Monte Video . .	Graving Dock	1	55 0	Footway	9 6	„ „	217	„

PONTOONS.

Dublin . . .	Royal Canal	1	18 6	Highway	52 0	One piece	18	Manual.

TABLE OF MOVABLE BRIDGES (*Continued*).

Locality.	Water Crossed.	Number of Passages.	Width of each Passage.	Nature of Way over.	Width of Bridge over all.	Number of Parts.	Total Weight of Moving Parts.	Moving Power.
			Ft. Ins.		Ft. Ins.		Tons.	
			PONTOONS—(*continued*).					
Toulon . . .	Canal Mangegarry	1	85 0	Highway	16 6	Two pieces hinged together	..	Manual
Calcutta . .	River Hooghly	1	200 0	,,	..	Two pieces	..	,,
			LIFT BRIDGES.					
Dublin . . .	Spencer Dock, Royal Canal entrance	1	14 6	Railway	12 0	One piece	14	Water balance and manual.
London . . .	Grand Surrey Canal	1	21 0	,,	23 6	,, ,,	$12\frac{3}{4}$	Manual.
Dublin . . .	Royal Canal	1	14 6	,,	12 0	,, ,,	$11\frac{3}{4}$	,,]

CHAPTER XXII.

TUNNELS.

General Arrangement.

The importance of tunnels as a means of communication is so evident as to require no insisting upon in this place. The attention of the engineer was early devoted to their construction, and they have afforded a field for the development of some of the greatest proofs of his skill.

Before any decision can be arrived at by the engineer as to the course, levels, dimensions, and mode of construction to be adopted, careful surveys and examination of the strata geologically are requisite, together with levels or soundings from which a profile or section of the surface of the ground to be passed under may be made. The geological character of the strata must be ascertained either by borings, by sinking trial shafts or pits on the line of the intended tunnel, or by a small driftway or heading nearly following its course. The trial shafts thus sunk serve afterwards as working shafts through which the earth from the tunnel can be raised during its progress, and, by working either way from them, enabling the work to be carried on with great rapidity by breaking up the tunnel into short lengths, and, on the completion of the work, affording the means of light and ventilation. The advantage of a driftway consists in the more perfect drainage of the ground through which the tunnel is to be formed, and its insuring the certainty of the tunnel being formed correctly on the line intended. The nature of

the various strata to be passed through having been accurately ascertained by one or other of these means, the engineer will be enabled to determine upon the best form of transverse section to be adopted for the tunnel, and upon the thickness of the masonry requisite to support securely the sides and roof; and he will be further enabled to judge of the probable amount of water to be met with, and to make such preparations, by the provision of proper and sufficient means of withdrawing it, as to prevent delays in the progress of the work. He will also, by such a particular acquaintance with the nature of the ground, be enabled to devise the simplest mode of securing the ground temporarily during the construction of the tunnel, and of anticipating and preparing for any difficulties arising from the varying character of the soil.

The subjoined table exhibits the principal dimensions of some of the most important tunnels which have been constructed, together with the general nature of the ground through which they were made, the name of their engineer, the materials of which formed, and their cost.

TABLE OF THE DIMENSIONS OF SOME OF THE PRINCIPAL TUNNELS.

Reference to Woodcuts.	Name and Situation of Bridge.	Purpose to which applied.	Extreme Height.	Extreme Width.	Thickness of Lining at Crown of Arch.	Length.	Nature of Strata.	Material of Lining.	Total Cost.	Cost per Yard forward.	Date of Completion.	Name of Engineer.
			ft. in.	ft. in.	ft. in.	Yards.			£	£		
	Thames and Medway Tunnel.	North Kent Railway.	39 0	35 6	1 2	3960	Chalk and fullers' earth.	Brick-work.	..	..	1800	W. T. Clarke.
	Islington Tunnel.	Regent's Canal.	21 6	20 0	1 6	900	London clay formation.	Ibid.	..	..	1812	Morgan.
	Harecastle Tunnel.	Tetney Haven Canal.	16 2	17 0	1 2	2926⅓	Various strata.	Ibid.	112,681	38⅓	1827	Telford.
	Watford Tunnel.	North Western Railway.	26 6	27 0	1 6	1830	Chalk.	Ibid.	..	..	1838	R. Stephenson.
	Box Tunnel, near Bath	Great Western Railway.	36 6	36 0	2 3	3123	Freestone rock.	Ibid.	..	..	1838	I. K. Brunel.
	Littleborough Tunnel.	Manchester and Leeds Railway.	27 6	27 0	1 10½	2860	Various strata.	Ibid.	251,000	88	1841	G. Stephenson.
	Under the Thames at London.	Foot passengers.	22 3	37 6	2 6	400	London clay formation.	Ibid.	454,714	1137	1842	Sir I. Brunel.
	Blechingley Tunnel.	South Eastern Railway.	30 0	30 0	1 10½	1824	Shale.	Ibid.	95,237	71	1842	W. Cubitt.
	Saltwood Tunnel.	Ibid.	30 6	30 0	2 3	954	Lower green sand.	Ibid.	112,642	118	1843	Ibid.

Fig. 153 is a transverse section of the tunnel constructed originally for the Thames and Medway Canal, but now used for the North Kent Railway; Fig. 154 is a section of the tunnel carrying the Regent's Canal through Islington; Fig. 155 a section of the Harecastle Tunnel on the Tetney Haven Canal, made by Telford, to take the place of a smaller

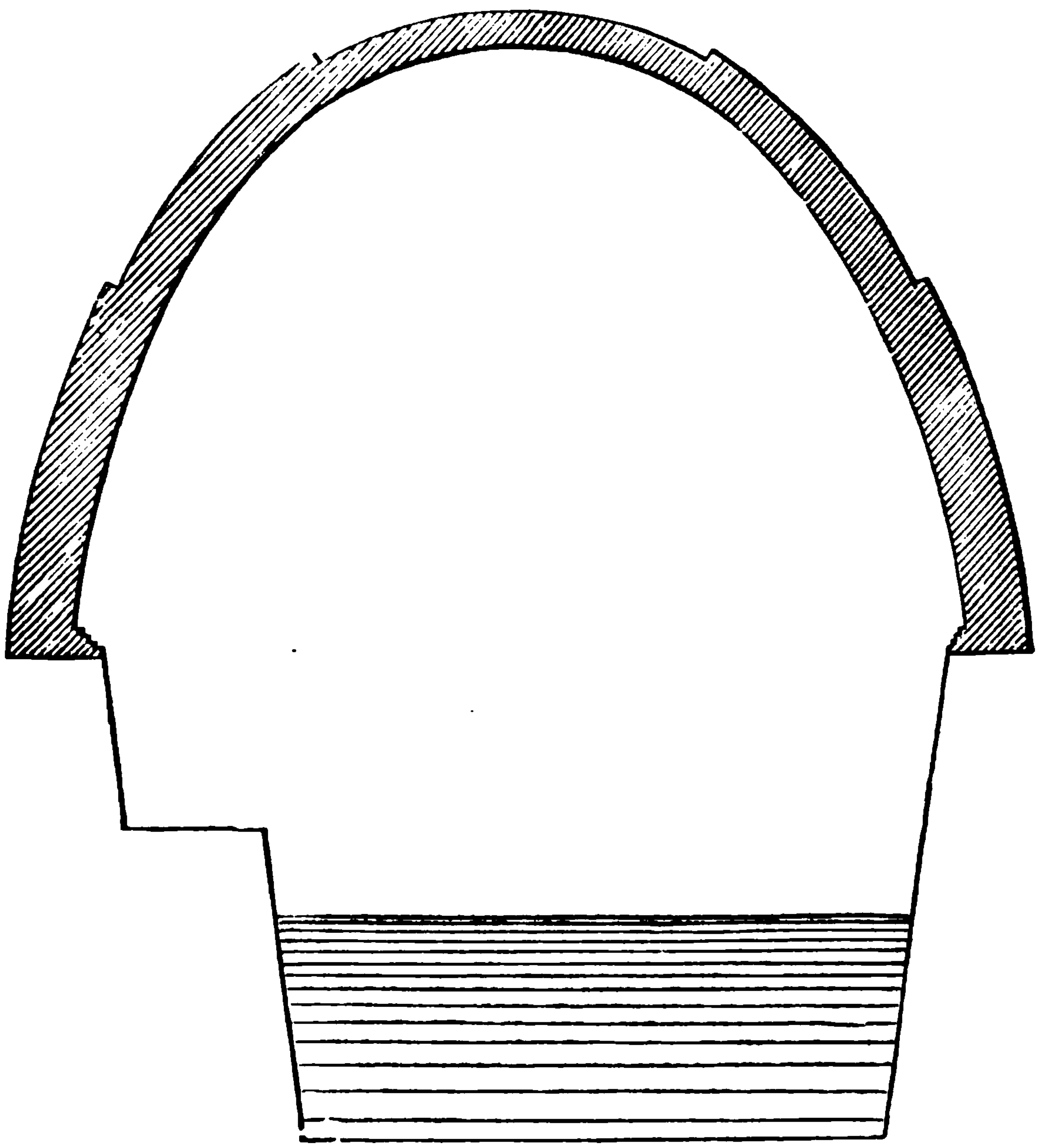


Fig. 153.—Tunnel, Thames and Medway Canal.

one previously made by Brindley; Fig. 156 is a section of the tunnel for the North-Western Railway at Watford; Fig. 157 is a section of the tunnel near Bath, for the Great Western Railway; and Fig. 158 is a figure of the Blechingley Tunnel, on the South-Eastern Railway. The transverse section of the Saltwood Tunnel is precisely similar to the

last, with the exception of being 6 inches greater in height.

Fig. 154.—Tunnel, Regent's Canal. Fig. 155.—Tunnel, Tetney Haven Canal.

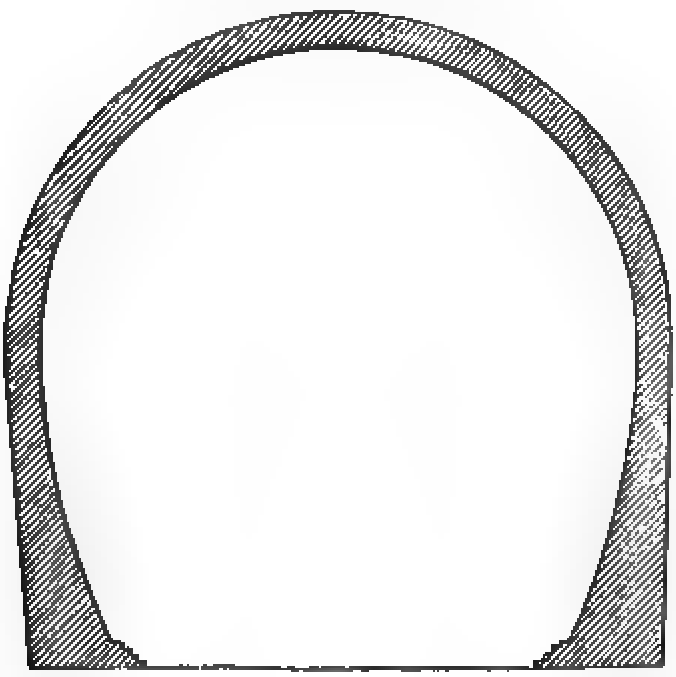


Fig. 156.—Railway Tunnel, Watford, London and North Western Railway.

The whole of the foregoing sections are drawn to a uniform scale of 12 feet to the inch.

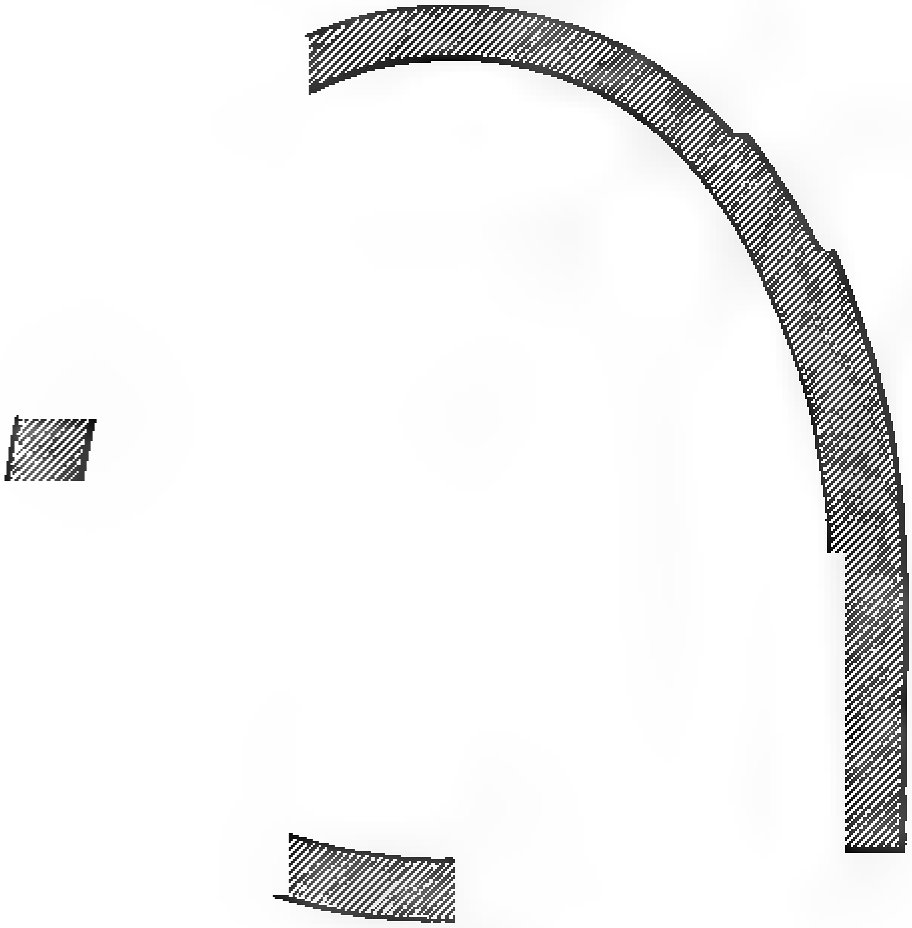


Fig. 157.—Railway Tunnel, Bath, Great Western Railway.

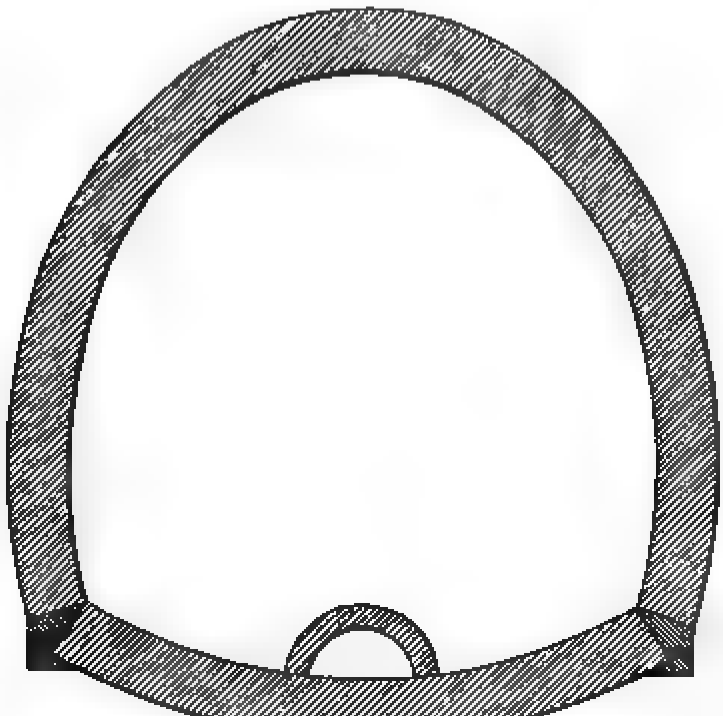


Fig. 158.—Blechingley Tunnel.

MODE OF CONSTRUCTING ORDINARY TUNNELS.

Having determined upon the exact course of the tunnel, the next point is to arrange the position of the shafts. These are best placed at equal distances, and their frequency should depend upon the time in which it is necessary to complete the work. There is, however, a certain distance in every case which will be more economical than any other, and this will be readily understood if we bear in mind that the cost of the tunnel itself per foot forward becomes greater as its distance from the working shaft increases, so that by lessening the distance between the shafts and increasing their number we diminish the cost of the tunnel itself; when, however, the shafts are placed too close, their cost becomes greater than the saving upon the tunnel, and there will therefore, in every case, be a certain distance, depending upon the relative cost of the tunnels and shafts, at which the expense of the whole work will be a minimum.

There are two methods in ordinary use for sinking the shafts: the first can only be followed when the ground through which it has to be sunk is tolerably firm and free from water, and consists in making an open excavation of the form and dimensions of the shaft, including space for the internal lining of brick or other materials, and to such a depth as the nature of the ground may indicate to be safe. A ring or *curb* (as it is technically called) of timber is then laid on the bottom of the excavation, previously levelled to receive it. This curb is formed either of one thickness with lapped joints, or in two thicknesses breaking joint (as shown at A A, Figs. 159 and 160) securely bolted together. The thickness of the curb should depend upon the dimensions of the shaft, being in no case less than 3 inches; its internal diameter should be the same as that of the shaft, and its breadth may be made greater, as shown in the figures, so as to project into the ground and assist in supporting the struc-

ture. As the curb becomes a part of the permanent work, it should be of oak or elm timber of the best quality. The

Fig. 159.—Shaft of Tunnel.

curb being placed, the wall or lining (B B) of the shaft

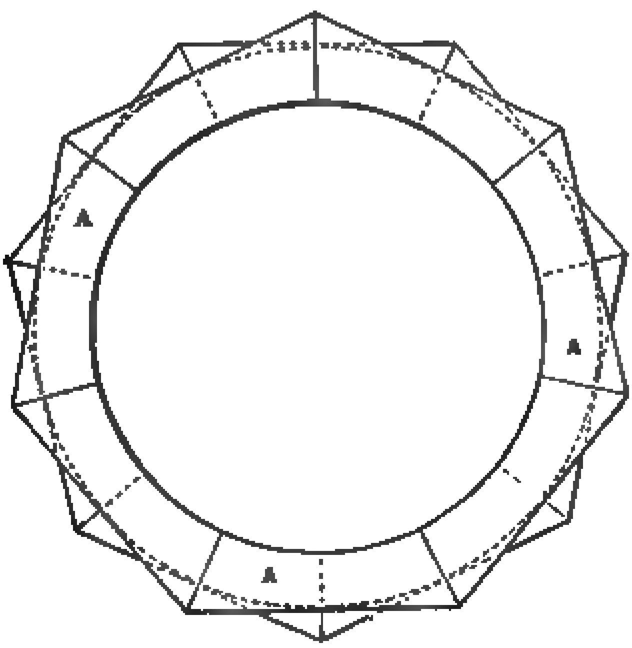


Fig. 160.—Curb for Building Shaft.

should be proceeded with, especial care being taken to ram in the ground firmly on the outer side, so as to leave no

space or vacuity: indeed it is impossible, in all operations in tunnels and other subterranean works, to pay too much attention to prevent the slightest vacuity between the work and the ground; but, on the contrary, whenever the ground is at all loose or disposed to move, every inch of surface should be well supported, and not only supported, but well strutted against, so as to maintain an active pressure at all times against it. As soon as the brickwork forming the lining has been carried up to the level of the ground, and

Fig. 161.—Shaft of Tunnel.

the earth securely rammed or *punned* in behind it, the excavation for a second length may be proceeded with. This, however, must be done with caution, so as not to endanger the stability of the portion already built by undermining its foundation. We must first carry down the excavation in the centre of the interior of the shaft, leaving sufficient ground under the curb safely to support it; we may then cautiously remove the ground from under the curb at four

opposite points, leaving the intermediate ground to form piers for its support. The spaces or recesses thus excavated afford the means of introducing shores or props for the temporary support of the curb while the remainder of the ground is being removed. These props should be placed in an inclined position, as shown at C C C, Fig. 161, so as not to be in the way of the second curb; they should be spiked to the upper curb, to secure them from slipping out of place, and should rest at their lower extremity upon a broad sole piece, D D D, to prevent their sinking into the ground. The props having been introduced, the remainder of the ground may be removed, a second curb, similar in every respect to the former, laid at the bottom of the excavation, and the lining of brickwork proceeded with, in the spaces between the timber struts, in the manner shown in Fig. 161. Upon the brickwork being brought up to the under side of the first curb, great care should be taken in perfectly filling up the space, so that the curb may have a firm and secure bed upon the brickwork below it. The props or struts may then be removed, and the brickwork completed in the spaces which they had occupied. The excavation should be again proceeded with, and the various operations already described repeated until the shaft has attained the required depth. The mode of building shafts which has just been described is technically termed *underpinning*.

The second method is frequently employed in sinking wells, and must always be adopted when the soil is too loose or full of water to allow of an open excavation being made with safety. It consists in forming the curb as shown at A A, Fig. 162, with a sharp edge or rim, instead of having a broad flat surface, as in the former case; upon this curb the brickwork of the shaft is to be built as before until carried up to the level of the surface. The excavation within the shaft is then to be proceeded with, the whole of the ground being in this case removed from under the curb,

which, being thus left without support, and being loaded with the weight of the brickwork upon it, will gradually descend; and thus, as the excavation is carried down, the curb will follow, and as it sinks the wall must be carried up, so as to maintain it level with the surface of the ground. The principal care required in this mode of sinking shafts is, to avoid one side of the curb descending more rapidly than the opposite one, by which the shaft would be thrown out of the perpendicular, and so much resistance occasioned as possibly to prevent its further descent. By a little management, however, in the removal of the ground from beneath

Fig. 162.—Shaft of Tunnel.

the curb, this may be usually avoided; and, when earth-bound, the shaft may frequently be set free again by pouring water around it, so as to soften the ground on its outer side. A very good precaution against a shaft becoming earth-bound is to build it slightly tapering upwards; this tapering, however, should not be too considerable, otherwise the space left around it by the descent of the shaft would be sufficient to loosen and dislocate the surrounding ground.

The brick shaft having, by one or other of these means, been carried down to within a few feet of the top of the intended tunnel, the excavation should then be cautiously

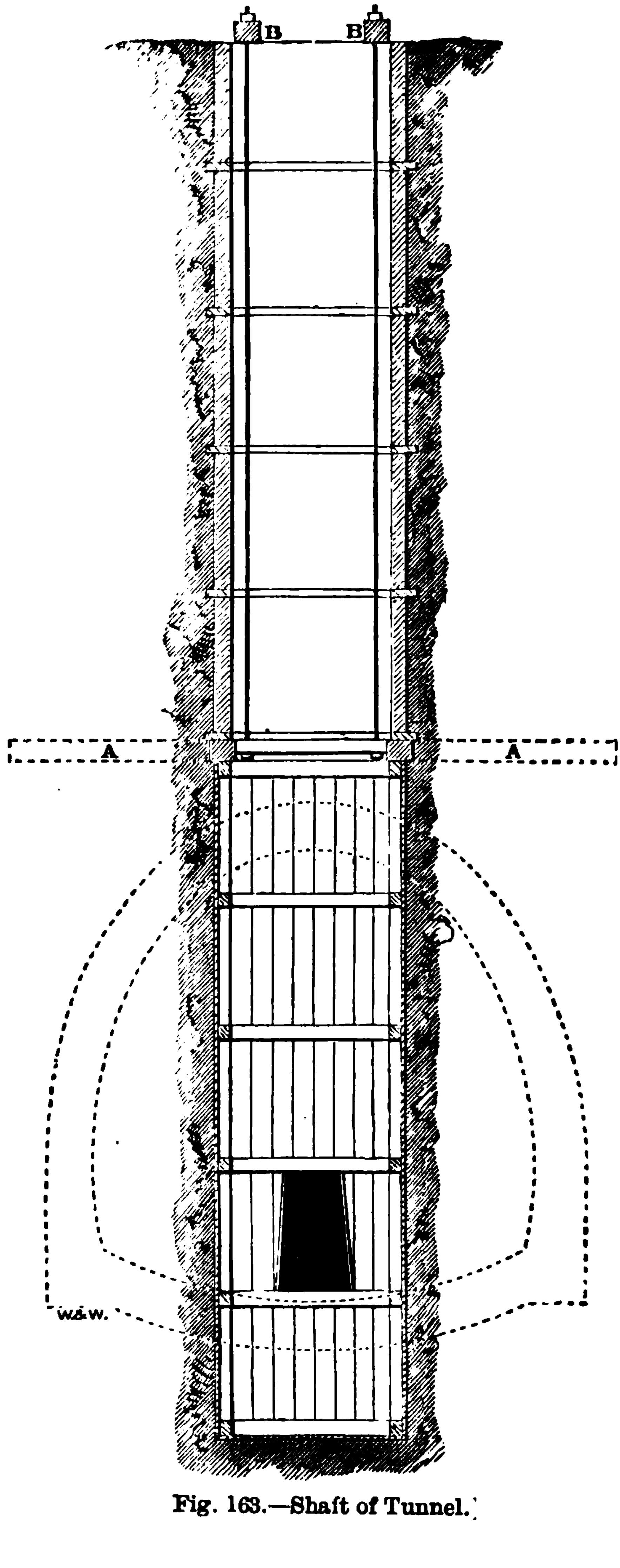


Fig. 163.—Shaft of Tunnel.

proceeded with, the sides being secured with timber framing and planks, until carried below the level of the bottom of the tunnel. Particular care should be taken that no movement in the ground takes place, because the difficulty of forming the tunnel would be greatly increased if the ground through which it had to be formed had been previously disturbed. The manner of securing the lower portion of the excavation with timber is shown in Figs. 163 and 164, the former being a vertical, and the latter a horizontal section. Previously, however, to carrying down the excavation below the brickwork of the shaft, some means must be adopted for its support, as the mere friction of the ground against its exterior surface would not be sufficient to sustain its weight. It is therefore necessary either to support it by introducing timbers underneath it, as shown at A A, in Fig. 163, or to suspend it by rods secured to timbers resting on the surface of the ground, as shown at B B.

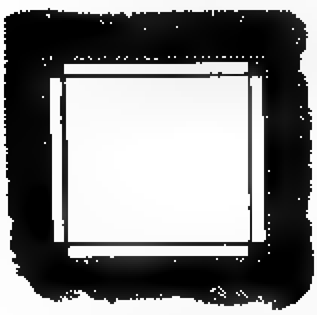


Fig. 164.—Shaft.

A small driftway or heading should now be commenced about the level of the bottom of the tunnel, and having a sufficient inclination given to it to enable any water met with to drain into the bottom of the shaft, which thus becomes a well or sump for the drainage of the works, and from which the water may easily be removed by any of the usual methods. The dimensions of the heading should be sufficient to enable a man to work in it without inconvenience. The usual size is about 3 feet wide and 5 or 6 feet in height; its sides should be secured with timber, as shown in the transverse section Fig. 165.

Fig. 165.—Heading.

Fig. 166 is a longitudinal section of a portion of a tunnel in progress of construction. *c* is the driftway, which has been carried forward to meet that from the next shaft, so as to form a continuous means of communication from shaft to shaft, the importance of which is considerable in enabling the direction and level of the tunnel to be set out without chance of error.

These preliminary works having been completed, and the form and dimensions of the tunnel determined, the excava-

Fig. 166.—Excavation of Tunnel.

tion for it must be commenced, the ground being supported by means of timber and planks in the manner shown in Figs. 166 and 167. The longitudinal timbers, *a a*, are termed *bars*, and the transverse planks, *b b*, *polings*. The length thus excavated at a time must depend upon the quality of the ground; but as it is always desirable that the surface of the ground should be exposed to the atmospheric influence for as short a time as possible, it is not proper to proceed too far before inserting the brickwork.

To insure the brickwork of the tunnel being true in form,

curved templates are used for the invert and sides, while the upper portion is turned upon a centre similar to those employed for turning the arches of bridges. As the brickwork proceeds, the bars and polings must be carefully removed, and any vacuity thus left must be filled with earth well rammed in, so as to prevent any settlement of the ground, which would occasion unequal strains upon the body of the tunnel. As in most strata some amount of settlement will take place in the superincumbent ground before the brick-

Fig. 167.—Excavation of Tunnel.

work can be got in, the timber and polings should be placed a few inches above the top of the tunnel.

As soon as a length of brickwork has been got in on each side of the shaft, the temporary timber work of the lower portion of the shaft should be carefully removed, the ground excavated to the true form of the tunnel, and the brickwork introduced; being securely bonded with and connected to that already built on either side, and the brickwork of the shaft being carried down to meet that of the tunnel. When

this has been properly effected, much of the danger and difficulty of the work may be considered as having been surmounted. The excavation of the face of the work must then be proceeded with, the top and as much of the sides of the work being supported by polings as may be found necessary, especial care being taken to prevent any disruption or movement of the ground. When the faces of the two opposite workings approach within a short distance of each other, great caution is necessary to avoid the thin partition of ground being disturbed. When sandy or other loose strata containing large quantities of water are met with, peculiar precautions must be taken to prevent the loose ground being washed in with the water, which would occasion cavities to be left in the surrounding ground. It would, however, be equally dangerous to dam back and confine the water, and therefore such means must be resorted to as will permit the water to percolate into the work, but prevent the ground being brought with it: a very simple and effectual mode, under ordinary circumstances, is to thrust straw into any opening from whence muddy water is found to proceed.

Upon the completion of the tunnel it is desirable that the shafts should be kept open, to afford light and the means of ventilation; but, in order to avoid accidents, it is advisable to carry up the brickwork to a height of 8 or 10 feet above the surface of the ground, and to cover the opening or mouth with a strong iron grating.

Should the strata through which the tunnel has been constructed contain much water, a certain portion will be found to penetrate the brickwork, however carefully built, and in such a case a small drain or culvert should be formed along the centre or lowest part of the invert.

Mode of constructing Subaqueous Tunnels.

We shall now proceed to describe generally the mode adopted in the construction of the tunnel under the Thames

between Rotherhithe and Wapping. The form of the tunnel

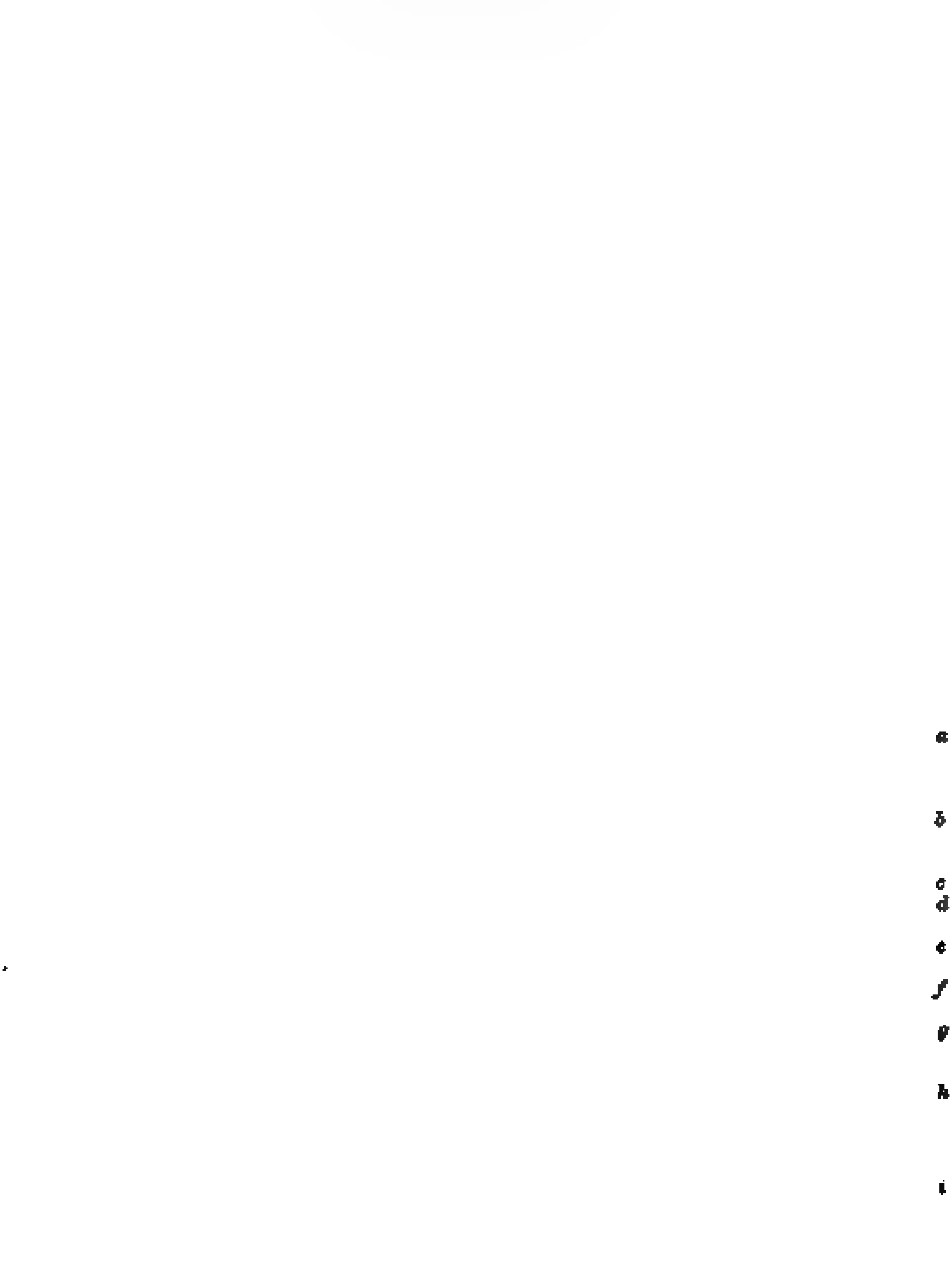


Fig. 168.—Thames Tunnel.

will be understood by a reference to Fig. 168, from which it

will be seen that the external form is rectangular, the reason for which was that the strata being horizontal, and, from their proximity to the river, subjected to constantly varying pressure, it was considered that a circular structure would have been exposed to very irregular strains. The archway was made double, in order that carriages might not have to meet and pass in the same opening, and the centre or partition wall thus formed was pierced with frequent arches, as shown in the longitudinal section, Fig. 169, which serve as a means of communication between the two archways, and form a very pleasing architectural feature in the tunnel.

Fig. 169.—Thames Tunnel: Longitudinal Section.

The external dimensions of the excavation are—in height 22 feet 8 inches, and in breadth 37 feet 6 inches, its total length is about 1,200 feet. The height of the archway is 17 feet, and the width of each on the springing line 14 feet; the upper portion is semicircular in form, and the side walls and invert segmental. The tunnel is built principally in half-brick rings, the thickness of the brickwork at the crown of the arch being 2 feet 6 inches, and the same below the invert, which is laid upon 8-inch elm planks. The external piers are each 8 feet thick on the springing line, and the centre pier is 3 feet 6 inches. The left-hand half of the section, Fig. 168, exhibits the mode in which the

bricks were arranged when working in 9-inch rings, and the right-hand half when $4\frac{1}{2}$-inch work was employed. The tunnel was built of the hardest picked stock bricks, laid in Roman cement of the best quality, those portions of the work which were most exposed to the action of the water being laid in pure cement, and the other portions in half cement and half pure sharp sand. The bricks for the semicircular arches are made in a wedge form, so as to produce parallel joints.

The section, Fig. 168, is taken in the centre of the tunnel, in the deepest part, showing the order and position of the various strata met with, as they would have been found if they had not been disturbed; from the constant runs of loose sand and the action of the water, especially on the Wapping side of the river, the strata, however, were usually found considerably dislocated and disturbed. In the section, *a* is a stratum of sand, gravel, mud, and river deposits; *b*, a bed of clay, of a reddish-brown colour; *c*, a stratum of clay mixed with silt; *d*, a thin layer of silt very full of shells; *e*, a stratum of stiff blue clay; *f*, a bed of clay of a more mottled character, containing a portion of silt and a number of shells; *g*, a stratum of indurated clay, which at times was so hard as to require wedges to break it up; *h*, a bed of gravel and sand of a green colour; and *i*, a similar stratum, but somewhat coarser.

The tunnel was commenced on the Rotherhithe side of the river in the year 1826, the shaft having been begun early in the previous year. The mode adopted in the sinking of the shaft was similar to that described, the brickwork being built upon a sharp-edged curb, which descended gradually as the ground was removed from under it. When, however, the shaft had thus been sunk to a depth of 38 feet, it became earth-bound, and, although loaded with a considerable extraneous weight, and the water allowed to rise in the excavation for the purpose of softening the ground, no further

movement took place; it was therefore determined to complete it by underpinning in the manner already described, and this, after much trouble and difficulty, arising from the loose nature of the ground, was successfully accomplished. When the shaft was sunk on the opposite or Wapping side of the river, the difficulties which had been encountered in the sinking of the former one were provided against, and the operation so successfully performed that the shaft was sunk to its entire depth (upwards of 72 feet) without becoming at all bound; this was principally owing to the shaft being made larger in diameter at its lower end, and the cast-iron curb being made of great strength.

Our limits will not permit, and the object of the present work does not require, our giving a detailed account of the many casualties and difficulties which were experienced in the course of the work; we shall content ourselves with a general description of the mode in which the tunnel was constructed, and refer the reader who requires further details to the "Memoir of the Thames Tunnel," in 4to.

The mode of securing the ground, already described as being employed in the case of ordinary tunnels, would have been quite inadequate to support the ground and the excessive and constantly varying pressure occasioned by the river. To meet the special requirements of the case, Sir Isambart Brunel devised a machine, constructed entirely of iron, and so original in its character as to enable him to secure the invention by letters patent. It occupied a space of about 8 feet in advance of the brickwork, and consisted of twelve distinct frames, each about 3 feet in width and 22 feet in height, ranged side by side, like the books on the shelves of a bookcase. Each of these frames was divided vertically into three cells by cast-iron floor-plates, so that the whole shield consisted of 36 cells. The roof and sides were secured by a number of narrow plates of metal, overlapping the portion of the brickwork already built, and entering the

ground in advance of the work; while the face of the excavation was secured by timber *polings*, so accurately fitted as to leave no aperture whatever through which loose strata could find their way.

The following brief description of the shield, and the mode of using it, is extracted from Weale's "London Exhibited:"—

"It will at once be seen how admirably the shield was adapted for the duties which it had to perform; the chief of these was, obviously, to support the ground, but a quality equally essential was, the power of being easily advanced or moved forward, as the tunnel progressed. Now, by its division into frames, these two objects were at once attained, for the whole was so contrived that, while six alternate frames were engaged in sustaining the pressure of the ground, the six intermediate frames were relieved entirely from all pressure, and left free to be moved forward without resistance. These, in their turn, then became the pressure-bearers, relieving those which had previously relieved them in a similar manner, and enabling them to be advanced without difficulty.

"'It has been already said that the shield, as first designed by Sir Isambart, bore a considerable resemblance to the worm from which the first idea was derived; but the present shield has much more aptly been compared with a man, to whom, in its general organization, each of these "frames" or divisions bears a resemblance; having legs with both a knee and ankle-joint, with which it alternately steps or walks on in advance of the brick structure; arms, with which it supports and steadies itself, or lends assistance to its neighbours when they require it; and a head, for supporting the superincumbent earth, which can be raised or depressed, or altered in its direction, as circumstances may require.' *

"Fig. 170 affords a view of the three left-hand frames of

* "A Memoir of the Thames Tunnel," in 4to.

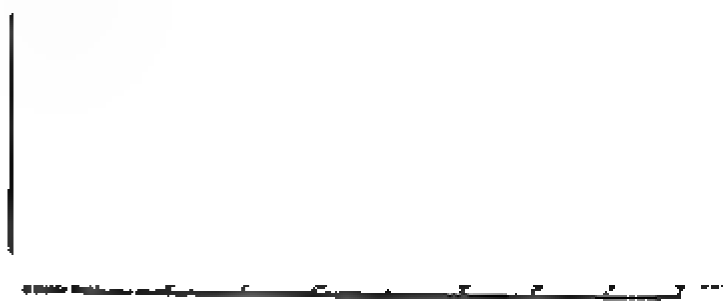


Fig. 170.—Thames Tunnel: Shield.

the shield, as seen from the tunnel, the third frame being shown in section, in order that the mechanism may be more clearly seen; and Fig. 171 is a section taken through the same frame, in a line parallel with the direction of the tunnel, or perpendicular to that shown in Fig. 170. The sides of the boxes, or frames, are formed by strong castings, A A, securely bolted to the floor-plates, B B, which, as already explained, served to separate every frame into three stories, or boxes. The middle boxes were stiffened, both transversely and longitudinally, by wrought-iron stays or struts, C C and D D; and the shield was strengthened at the back by two wrought-iron straps, E E, which extended from the top to the bottom of both sides of each frame, passing through the intermediate floor-plates. The framings of the upper and lower boxes were sloped away at the back, as shown in Fig. 171, to allow more room for the bricklayers in putting in the brickwork. The lower part of the bottom box was secured by a wrought-iron stay or framing, F and G, and the upper part of the top box by two similar framings of wrought iron, H and I. Each frame was supported upon two long jack-screws, K K, which, from the duty they had to perform, were termed *legs;* the lower extremities of these jacks rested upon strong wrought-iron plates, L L, termed *shoes*, whose object was to distribute the weight of the frames, together with the pressure of the superincumbent earth, over a large surface or base; beneath these shoes a flooring of elm planks, 3 inches in thickness, was laid, upon which the brickwork of the tunnel was built, after the ground beneath them had been compressed by the weight of the shield passing over them. The leg was attached to the shoe by a species of ankle-joint, *e*, resembling in principle the method adopted for mounting mariners' compasses, which allowed the shoe to adjust itself readily to any inequality in the ground. At the upper part of the leg was a knee-joint, M, about which it turned in the act of stepping forward: the length of the leg could be varied at pleasure, by means of the

Fig. 171.—Thames Tunnel : Shield.

screw at M, turned by the capstan-head at M, and a second auxiliary one in the middle box, N.

"The frames were also provided with slings, or arms, O, consisting of strong wrought-iron bars, attached at their upper extremities to the floor-plates of the odd-numbered frames, and at their lower extremities to the floor-plates of the even-numbered frames; the attachment consisting in an eye fitting to a circular pin projecting from the side of the floor-plates, so as to allow a freedom of motion about these pins as a centre. The upper and lower extremities of the slings consisted of two separate bars of metal connected by two plates or cheeks, one on either side, through which, and the slings themselves, metal keys or wedges passed, by the tightening up or driving back of which the length of the slings could be increased or diminished at pleasure. The use of these slings was to enable one frame to *derive* support from its neighbour on either side, or, in its turn, to *afford* support to either of its neighbours. Thus, if one of the odd-numbered frames, in which the upper extremities of the slings were attached to the top floor-plates, was required to be supported independently of the legs, it was only requisite to tighten up the wedges and lengthen the slings to raise the frame, and relieve the legs entirely from pressure; the slings, in this case, *pushing up* the frame. While, in the case of an even-numbered frame, by driving back the wedges of the slings on either side, and so lessening their length, the frame would be *drawn up*, and the legs relieved from the office of supporting the weight of the frame.

"The ground over the roof of each frame was supported by two plates of metal, Q Q, the tails of which always overlaid the brickwork, as shown in Fig. 171, and the points entered the ground some distance in advance of the boards, by which the front of the shield was secured. These plates of metal (which were technically termed *staves*) were supported upon a cast-iron saddle piece, R, resting upon a swivel, S, which

latter, being supported in front upon a kind of joint, U, and at the back upon a jack or strong screw, V, could be raised or lowered at pleasure. This mode of supporting the top staves allowed of their being brought into any position, or having any direction given to them. The tails of the staves were supported by a powerful jack-screw, W.

"The sides of the shield were secured, and the ground supported, by a number of similar staves, Z Z Z, Fig. 170, attached to the frames by a sliding bar, passing through a block secured to the sides of the external frames, in such a way as to allow of their direction being altered as circumstances might require. The tails of the side staves overlapped the brickwork of the tunnel in the same manner as the top staves.

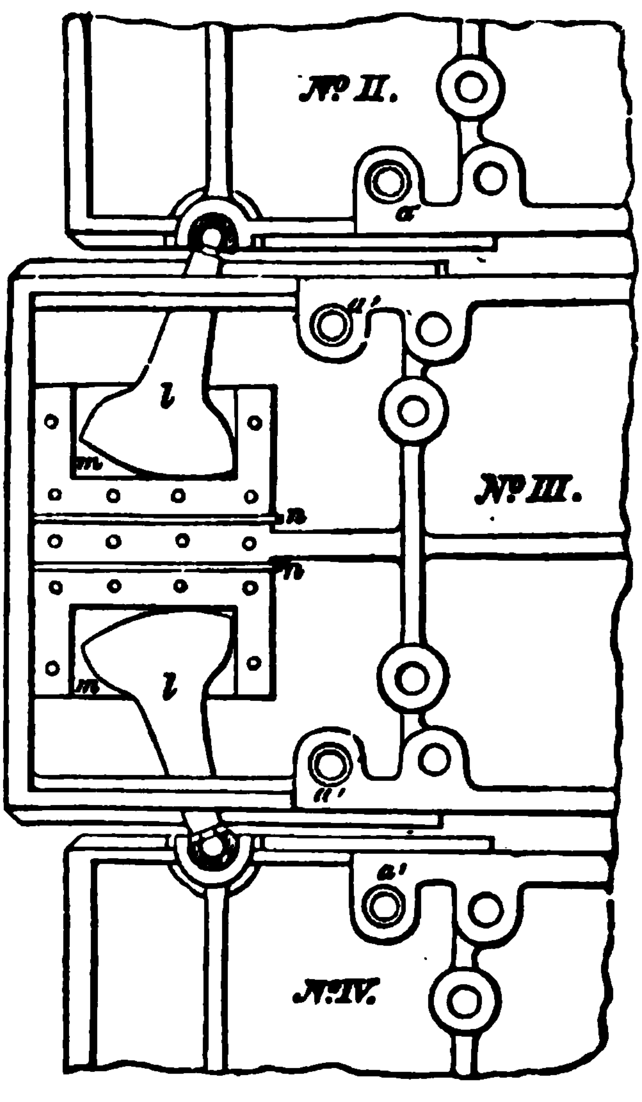


Fig. 172.—Shield Frames.

"The ground in front of the shield, as we have already mentioned, was supported by small boards of wood, *d d*, termed *poling boards;* each frame had its own set of *polings*, their length corresponding with the width of the frames. These boards were 3 inches in thickness, 6 inches in width, and at each end had small iron plates let in containing a recess, into which the head of a small jack, *e e* (termed the poling screws), fitted; the other end of these screws, resting in recesses formed for them, in the front rail of the cast-iron framing, A A, composing the sides of each box.

"The frames of the shield were not in actual contact,

a space of nearly 3 inches being maintained between them, to avoid the resistance which would have arisen from the friction of the frames if they had been allowed to rub against each other; and in order to preserve this space, the floor plates of every odd-numbered frame were provided at each end with a pair of wrought-iron sectors of circles, *l l*, Fig. 172 (or, as they were termed, *quadrants*), the heads of which bore against the door-plates of the even-numbered frames, and the circumference of which worked at the recesses *m m*, formed in the floor-plates of the odd-numbered frames for their reception. The quadrants served only to prevent the frames approaching too close: to obviate their spreading, a powerful tie, formed by two wrought-iron bolts, *t t*, Fig. 170, was attached to the two external frames.

"Each frame was supported and maintained in a vertical position by two powerful screws, *f f*, Fig. 171, termed the *abutment* screws, one at the top and one at its lower extremity. The heads of these screws rested against iron plates, *h h*, which served to throw the pressure occasioned by the screw over a larger surface of the brickwork. It was by means of these screws that the frames of the shield were advanced.*

"We now pass on to describe the mode in which the excavation was carried on and the shield advanced. We should first state, that every alternate frame of the shield stood three inches in front of the intermediate frames, which latter, when advanced, were moved forward six inches at a time, so as then to stand (in their turn) three inches in advance of the others. Thus the odd-numbered and even-numbered frames alternately stood in advance of each other.

* "It should be mentioned that two shields were employed in the construction of the tunnel. That which we have just described was the second, and contained several improvements which experience had pointed out. They were, however, identical in principle, and in their general mode of action."

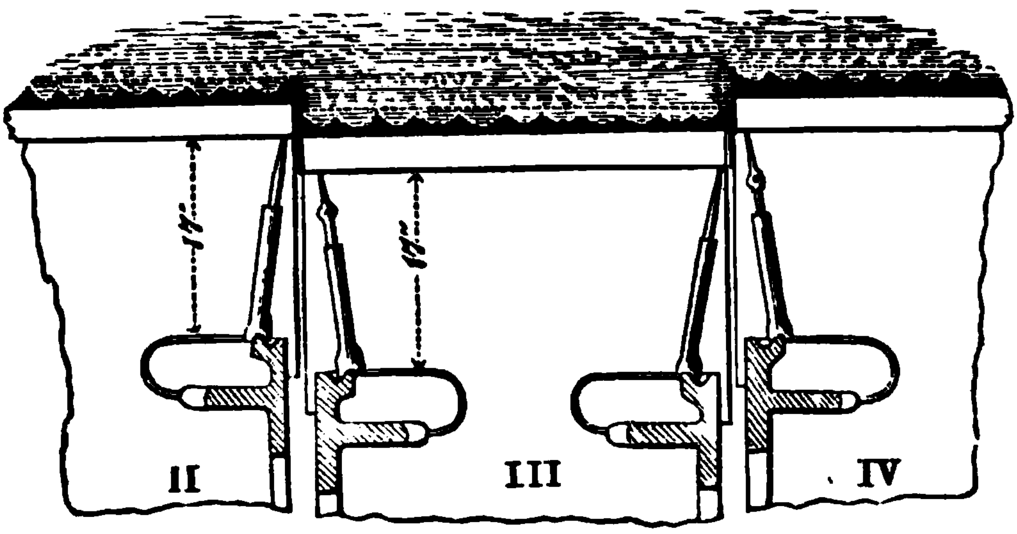


Fig. 173.

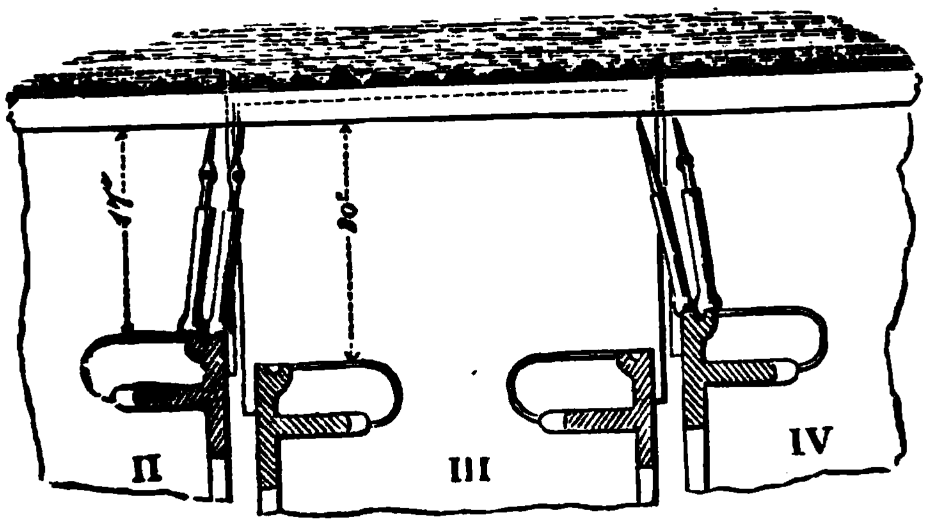


Fig. 174.

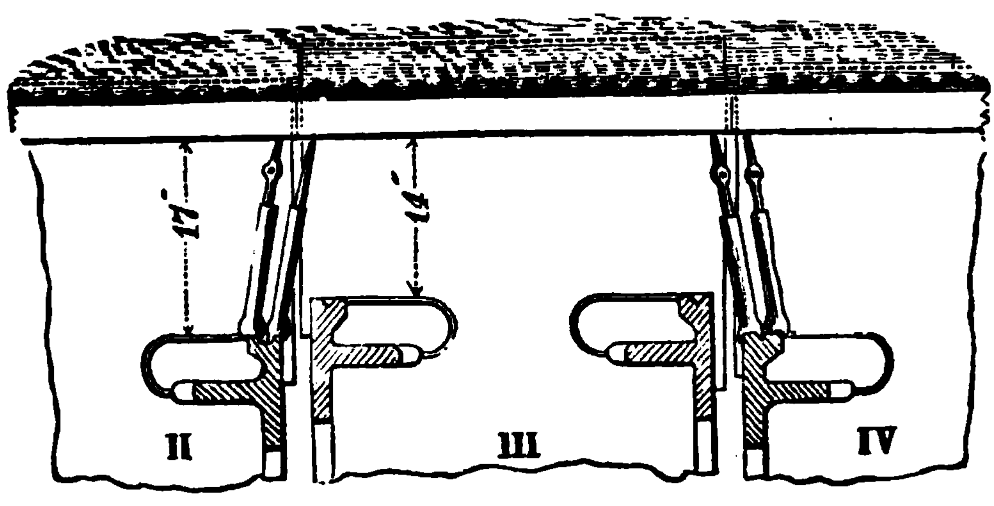


Fig. 175.

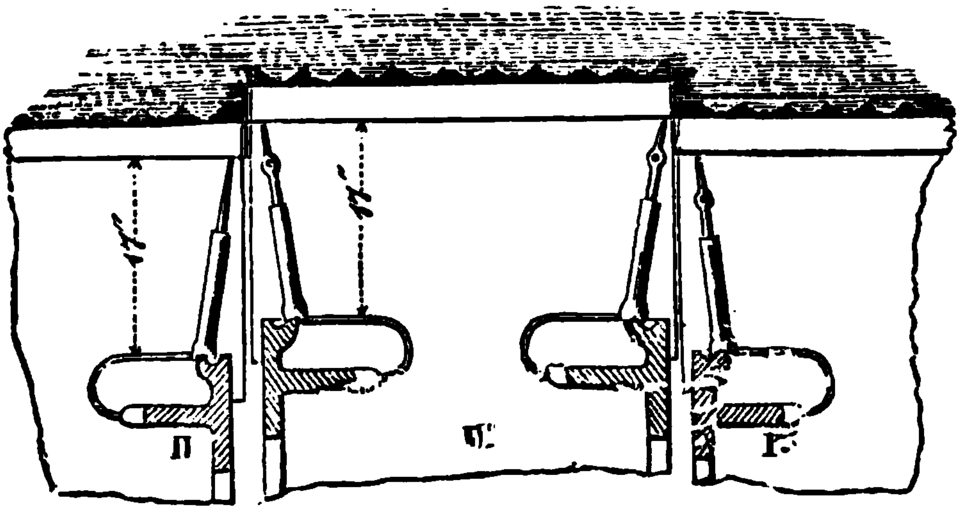


Fig. 176.

Thames Tunnel : Shield Frames.

We shall now suppose the odd-numbered frames to be behind, and proceed to detail the method of advancing one of them (No. III.), which will sufficiently explain the process adopted in the case of any one of the rest. Fig. 173 represents a section plan of a portion of the frames Nos. II., III., and IV., showing the relative positions of the front rails of those frames, together with their poling boards and the poling screws which supported them. This being the position of things, the first operation is, to remove the poling boards of the frame No. III., one at a time, commencing at the top of the box, and, having carefully excavated or cut away the ground to a depth of three inches, to replace the poling and its two screws; but instead of resting the latter upon their own frame, as they were before, they are now placed against the front rail of the two other frames on either side, as shown in Fig. 174; the object of this arrangement being, that the intermediate frame, after all the poling screws have been so removed, shall be left entirely free to be advanced or moved forward without experiencing any resistance from the ground against its poling boards, the whole of which are then temporarily supported by its neighbouring frames. The frame itself is then moved forward the required distance, or six inches, by means of the large abutment screws, *f f*, Fig. 171; the mode of operation being, first, to relieve the legs of the frame from weight by means of the slings, in the manner already explained, then to move forward the two shoes L L, Fig. 170, bringing the legs into the sloping position shown in Fig. 171, after which the frame itself is screwed forward by turning the upper and lower abutment screws simultaneously, until the legs are brought again into a vertical position, and the frame assumes the situation shown in Fig. 175, being then three inches in advance of its neighbours, Nos. II. and IV. The poling boards are now again removed, the ground once more

excavated to a further depth of three inches, and the boards and poling screws again replaced, the latter being again restored to their own frame, so that they assume the position shown in Fig. 176, the frames and polings of the odd-numbered divisions being now three inches in advance of the even-numbered frames, which latter, in their turn, will undergo a similar operation to that above explained.

"In Fig. 171 the polings in the upper box are shown as having been worked forward, while in the middle and lower boxes they are represented as being in the act of being worked; in the latter, two polings are shown out at once; this was usually allowed in the lower boxes, the ground in which, being further from the river, was usually more solid than in the upper boxes, and occasionally, when the ground in the latter was unusually good, the miners in those boxes were allowed also to remove two polings at a time.

"When the whole shield had thus been advanced sufficiently to admit of a ring of brickwork being introduced, this was immediately proceeded with, the arches being turned upon a narrow centering or profile, *v*, Fig. 171, and being inserted behind the abutment screws, *ff*, one at a time, care being taken that none of the poling screws were resting upon a frame whose abutment screws were not in proper bearing. As the shield advanced, a timber stage on wheels followed it, which afforded ready means of access for the miners and bricklayers to every part of the shield."

CHAPTER XXIII.

SYSTEMS OF DRIVING TUNNELS.

[Two systems of driving tunnels are recognised on the Continent—the English or German system, and the Belgian or French system. On the English system, so called, the main heading is opened and driven at the level of the floor or of the invert of the tunnel, and the excavation of the tunnel is developed upwards and laterally. On the Belgian system, so called, the main heading is opened and driven through the upper part of the tunnel, at such a level that the roof of the heading is level with and forms a portion of the roof of the excavation for the tunnel, whence the excavation is extended downwards. On this system, the arch is constructed before the walls.

In short, the bottom heading is said to be the basis of the English system, the top heading of the Belgian. This distinction, if there ever was a reasonable foundation for it, has long ceased to exist. The bottom heading, no doubt, was, in the early English practice of tunnelling, systematically employed, not only because it served as a drain for water, but also as a means of insuring accuracy in the levels and ranging or setting out of the work. The tunnels constructed on the Great Western Railway, between Bath and Bristol, in 1842—43, were commenced by the driving of a bottom heading from end to end before the enlargement was commenced. These tunnels were constructed through hard grey sandstone, and the work of enlargement from the bottom

heading was found to be costly and troublesome, more particularly as the excavated material was removed through the heading. There was the greatest difficulty in keeping the thoroughfare clear. The material in course of excavation fell on the road, and if it was not immediately removed it delayed the whole of the work of the tunnel by interrupting the circulation of the waggons. Where lining was to be applied to the tunnels, top headings were driven, and when the excavation for a length was completed, the lining was commenced.

In more recent practice, the bottom heading has not necessarily been constructed, either for purposes of drainage or for setting out. The tunnel-aqueducts for the Glasgow Waterworks were constructed without headings. They were cut at once to the full section, 8 feet by 8 feet. In boring one of these tunnels, which was 2,325 yards in length, twelve shafts were sunk from the surface, varying in depth—some of them 160 yards in depth. The twenty-four faces were cut simultaneously, and all the junctions were exact. Again, in the construction of the Netherton Tunnel, in 1856—58, the system of bottom heading, it is true, was employed, headings having been driven both ways from the bases of seventeen shafts, on the model of Simms's practice;* but, as a matter of fact, the enlarging and lining of the tunnel were in course of execution at each shaft before the headings were joined up; and so successfully had the setting out been effected, independently of the bottom headings, that no part of the tunnel diverged so much as one inch out of the direct line. It may be added that, since the tunnel, destined to form a portion of a canal, was of necessity on a level, a thorough bottom heading, if it had been made, would have been useless for purposes of drainage. Besides, the means of exhausting inflowing water by way of the shafts is, in all but exceptional cases, amply

* See Simms's "Practical Tunnelling," third edition, 1877.

sufficient for clearing the works of water. Exceptional cases arose in the construction of the Saltwood and the Buckhorn Weston Tunnels.

The Buckhorn Weston Tunnel, 789 yards long, was constructed in 1859—60, through Kimmeridge clay, on an incline. It was originally intended to sink only two shafts, and to utilise the incline to drain the tunnel, by means of a bottom heading driven from the lower end, through which the excavated stuff was to be conveyed. The heading was driven for a length of 200 yards, but it became so much contracted by the swelling of the sides that it was abandoned as a thoroughfare for water and waggons. The number of shafts was increased to five; top headings were driven from each shaft, and the tunnel was enlarged, timbered, and lined as in the Bletchingley Tunnel, described by Mr. Simms. In the course of excavation, however, veins of loose, rubbly rock were cut, from which large quantities of water were discharged, chiefly through the crown of the excavation. To check this inconvenient flow, a counter heading was driven from the end of the ridge, over the tunnel, for a distance of ten yards beyond the intersection of the vein, in which the top water was intercepted and drained off to the face. Here, instead of a bottom heading, a top heading was employed for drainage.

Whilst the common practice of sinking shafts and multiplying the number of faces possesses many advantages wherever expedition is an important element in the construction of a tunnel, the system is occasionally followed according to which a bottom heading is driven from the end at or near the formation-level, and a "break-up" is formed at intervals upon the heading. The break-up is a substitute for the excavation otherwise formed below the ordinary shaft, and the heading is equivalent in its functions to the shaft, constituting the means of communication between the working faces and the surface. But it is easily understood that

the employment of a single opening for the service of a number of faces cannot afford the facilities of free communication offered by several shafts sunk direct upon the tunnel at 200 yards apart, each of which is devoted to one pair of faces. The objections practically are similar in kind to those which were experienced in the construction of the tunnels on the Great Western Railway.

Moreover, bottom headings cannot be commenced until the cuttings or their gullets are run up to the faces of the tunnels; and when these are cut they are not in all situations available, as the inclination of gradients may not admit of drainage.

But the greatest disadvantage of the system of bottom headings and break-ups, in certain grounds, consists in the lengthened exposure of the surface of the excavation to the action of the air, which in clays, marls, and shales, loosens the ground, and in rock opens the fissures.

Finally, it may be urged as an objection to the system of working by break-up, that the means of ventilation is imperfect.

There are situations, nevertheless, where a break-up may be introduced with advantage. When a bottom heading is driven from the end of the tunnel, and a break-up is formed at some distance from the end, three working faces are obtained for each end, whilst the intermediate portions are worked from shafts in the usual way with top headings. The Lydgate Tunnel affords an instance of this method of procedure.

Comparatively short tunnels also, under 500 or 600 yards in length, may be economically constructed on the system of break-ups—with a through bottom heading, and, say, three break-ups in the whole length. Waggons run in on a line of rails under the working faces receive the broken stuff, which falls direct into them. But in situations where foul gases are likely to be emitted, the system of break-ups

is affected by the difficulty of the absence of sufficient ventilation, which may even prevent its being adopted at all.

In the history of the short tunnel, 500 feet long, on the Recife and São Francisco Railway, Pernambuco, constructed in 1856—62, an instance is afforded of the benefit that may be derived from a through bottom heading for the transport of stores. When the small trial heading had shown that the ground was favourable, as the materials for the construction of the permanent tunnel were not at hand, and there was plenty of timber on the ground adjoining, it was decided to enlarge the heading sufficiently to take an engine through, minus the chimney. This was effected without difficulty, the enlarged heading being 9½ feet wide at the bottom, 8 feet wide at the top, and 10 feet high. A constant stream of materials passed through during meal-times without interfering with the work. With the like objects, as soon as the end lengths had been completed in the usual manner, a surplus stock of materials, sufficient to last for a few days, was sent through. The road was then broken up, the invert was built throughout, and the road was relaid; after which the construction of the tunnel and the supply of materials went on without interfering with one another. The invert and arch of the tunnel were of brickwork, and the side walls were of rubble masonry.

When tunnels already constructed are to be enlarged—as, for instance, a railway tunnel constructed for a single line of rails, to be enlarged for two lines—the existing tunnel, it is obvious, serves as a ready-made bottom heading, and is so employed for the construction of the larger tunnel. The Lindal Tunnel was enlarged by raising shafts through the crown of the tunnel, making a break-up at each shaft, and driving top headings both ways from the break-up.

In tunnelling through rock, top headings are now usually driven, to be worked from. For example, the Clifton Tunnel, under the Durdham Downs, was cut through mountain lime-

stone, commencing with top headings from the shafts and a side gallery. The lower end was excavated by driving a bottom heading, from which a break-up with two faces was formed.

Other tunnels situated near the sides of mountains may be worked entirely from side drifts. The Shakespeare Tunnel, or, more correctly, double tunnel, Fig. 177, driven through the Shakespeare Cliff, near Dover, consists of two narrow tunnels, carrying each one line of rails, 12 feet wide and 30 feet in extreme height, through the chalk, separated by a solid pier or wall of chalk, 10 feet thick. The chalk was of variable quality, and the greater part of the tunnel was lined brick, strengthened by counterforts at 12 feet intervals. The tunnel is 1,430 yards in length, rising westward with an inclination of 1 in 264. The site of the tunnel being within a short distance from the face of the cliff, the material excavated was discharged through galleries about 400 feet long, driven in from the face of the cliff into the sea, the first operation having been to run a bench or roadway along the face of the cliff. There are seven shafts, averaging 180 feet deep.

Fig. 177.—Shakespeare Tunnel.

From the foregoing instances, which are types of many other tunnels, it may be inferred that there is no specifically English system. Top or bottom headings are used just as circumstances may direct. Mr. Simms, in "Practical Tunnelling," evidently wrote under the impression of the necessity of through bottom headings; but it has been amply proved by more recent experience that they are not necessary either for the purpose of effecting an exact setting out or for the purpose of drainage. With respect to the means of setting out a tunnel, it may suffice to mention that the Mont Cenis Tunnel, upwards of 6 miles in length, was set

out, with the aid, of course, of a through heading, entering from the ends. The junction of the headings was effected without any error horizontally, and with only a foot of divergence vertically. And as to the means of drainage, tunnels may in most cases be drained by means of steam-power through shafts; whilst tunnels like that of Mont Cenis, where either shafts or side-drifts cannot be applied, are constructed on inclines which rise from each end and meet at a summit in the interior, and so afford a fall for natural drainage.

In the excavation of tunnels of great length, and through mountains of great elevation, as, for instance, the tunnels under Mont Cenis and through the St. Gothard, where, in the absence of shafts, the only means of approach are from the ends, it is manifest that the proper method of penetrating the mountains is the method of the top heading, with subsequent enlargement. The excavation of the Mont Cenis Tunnel, it is true, was commenced by means of a bottom heading, from which the enlargement was effected upwards and laterally; but the system of the bottom heading has been completely negatived in the construction of the St. Gothard Tunnel, in which the top headings, or advanced galleries, were driven from the commencement, and continued to be exclusively followed in the excavation of the tunnel both in the solid rock and at the north end, and in the loose, uncertain, and watery

Fig. 176.—Tunnel under the Mound, Edinburgh.

strata of the south end. Drainage is more completely effected in rock by the method of the top heading, whilst also ventilation—a matter of importance in long tunnels—is more conveniently and effectively performed. For the excavation of these tunnels in rock, rock-drills driven by compressed air were employed.

The arches of tunnels are usually constructed of five or six rings of brickwork, making a total thickness of from 22 inches to 27 inches. The walls are of similar thickness.

Fig. 179.—Tunnel under the Mound, Edinburgh.

Inverts are constructed where the soil is not sufficiently strong to support the walls—that is to say, inverted arches built under the level of the rails.

The tunnel under the Mound at Edinburgh, on the line of the Edinburgh and Glasgow Railway, Figs. 178 to 180, supplies an excellent illustration of tunnels formed with an invert, in loose, unstable soil. The tunnel is truly circular, 28 feet in diameter and 20 feet high above the level of the rails, built of brick 36 inches in thickness, stiffened with counterforts externally, and with ribs of masonry internally, founded on a solid bed of coursed rubble work, with an inverted arch to distribute the weight. The mound stands 42 feet above the crown of the arch.

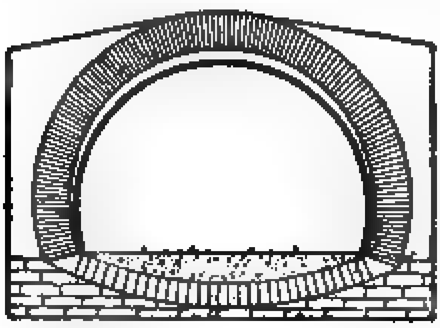


Fig. 180.—Section of Tunnel.

TABLE OF COST OF CONSTRUCTION OF VARIOUS TUNNELS, 1875.

Tunnel.	Railway.	Length.	Formation.	Cost per lineal yard.
		Yards.		£ s. d.
Blechingley.........	South Eastern	1,324	Clay	72 0 0
Saltwood	Do.	954	Sand	118 0 0
Buckhorn Weston	Salisbury and Yeovil	739	Clay	72 0 0
Lydgate	L. & N. Western	1,332	Coal Measures	80 0 0
Guildford............	South Western	965	Chalk	80 0 0
Salisbury............	Do.	440	Do.	80 0 0
Petersfield	Portsmouth	484	Do.	80 0 0
Honiton	Do.	1,350	Red marl and greensand	50 0 0
St. Catherine's ...	South Western	191	Sand	40 0 0
Clifton	Clifton Extension	1,737½	Limestone	...
Lindal (enlargement)	Furness	460	Rock, gravel and clay lined	21 4 0 38 0 0
Stapleton (do.) ...	Bristol and Gloucester	514	...	...
Netherton (canal)	Birmingham	3,036	Marl	39 5 0
Loch Katrine	Glasgow Water Works	2,325	Mica-slate	13 0 0
			Clay-slate	£9 to £10
			Old red sandstone	10 0 0
			Soft material, lined	10 0 0
Kilsby	L. & N. Western	2,398	...	125 0 0
Box	Great Western	3,123	Oolite, marl	100 0 0
Mont Cenis.........	Victor Emmanuel	7·60 miles	Calcareous schist, &c. estimated	167 12 0
St. Gothard.........	(In progress)	9·26 "	Granitic gneiss, &c.	116 9 0
Thames	East London	400 yards	Alluvial	1,137 0 0]

PART II.

MARINE ENGINEERING.

CHAPTER I.

WINDS AND WAVES.

[WORKS of marine engineering are broadly distinguished from other works of construction by the fact of their exposure to the action of the sea, whether at rest or in motion. The stability of such engineering works is largely dependent on the dead weight of the materials of which they are constructed, principally in order to counterbalance the pressure of the sea against them, augmented by the pressure and force of waves caused by winds and tides.

Winds—air in motion—acquire, as a maximum, a velocity of from 50 to 78 miles per hour, according to the quarterly weather reports of the Meteorological Department. The pressure of air in motion is very variously estimated. It is probable that the pressure of winds never exceeds 80 lb. per square foot; it appears to be well established that the maximum pressure in Great Britain does not surpass 24 lb. per square foot. In open spaces, winds blow downwards at a small inclination, and have the effect of raising the surface of water into waves. Dr. Scoresby, in 1850, stated, as the results of his observations, that in rather a hard gale ahead, with a heavy sea, the greater proportion of the waves

did not attain the height of 22 feet, although about one in four or five rose to about 26 feet or 28 feet in height. The average elevation was from 18 to 20 feet. In a hard gale with heavy squalls, most of the waves rose to more than 24 feet, and one in five or six rose to a height of 43 feet the pitch or distance apart of the waves was about 560 feet, whilst the velocity amounted to nearly 33 miles per hour.

The force of impact of a ground wave 20 feet high has been estimated by Mr. J. Scott Russell as equal to 1 ton per square foot of superficies. Even higher pressures than this have been registered. Mr. Alan Stevenson observed by dynamometer a pressure of 6,000 lbs., or nearly 3 tons, per superficial foot, at the Skerryvore lighthouse, when the waves broke, between high water and low water. A remarkable piece of evidence of the force of great waves occurred at the breakwater at Wick. A large monolith of cement rubble, 45 feet long by 26 feet, by 11 feet thick, weighing upwards of 800 tons, was gradually slewed round, by successive strokes, until it was finally removed and deposited inside the pier. It carried with it some courses of stone which had been fastened to it by iron bolts, the enormous mass weighing not less than 1,350 tons, and presenting an area of about 496 square feet to the force of the sea. The power of waves is also manifested by the lifting up of large blocks of coursed masonry from the paved slope of Plymouth Breakwater, and the throwing of them over on to the back slope; and by the piling up of the huge moles of shingle that fringe the southern and eastern shores of England, in the most remarkable of which, the Chesil beach, stones of great size are heaped up 35 feet above high water, or three times the usual height of such formations.

The tides, or oscillations of the surface of the sea, take place twice in twenty-four hours. Over the large areas of the Atlantic, Pacific, and Indian Oceans, the rise and fall due

to the tides is limited to from 2 to 4 feet; but it is augmented as the tidal wave approaches land and breaks on the shore. Compressed between projecting headlands, the tidal wave mounts in height; or when it is trapped between a great projection and its corresponding height; or led up some funnel-shaped deeply-indented inlet open to the wave; or in passing through straits between the main land and an outlying island, as in the Straits of Magellan, on the Patagonian coast, where the tide rises from the normal height of 4 feet to the extraordinary elevation of 36 feet. All our great estuaries, harbours, and tidal rivers which penetrate into the heart of the country are constantly fed by the tidal wave, the influence of which may be largely developed and increased by judiciously planned works of engineering. It may easily be comprehended that the height of the tide varies for every locality.]

CHAPTER II.

CURRENTS.

The stability of works erected upon the sea-shore is often affected, directly or indirectly, by the action of currents; and the remarkable influence they possess upon the direction of alluvions, sand, or shingle, renders their study the more indispensable. Some of the most striking illustrations of the importance and nature of currents are subjoined.

In the Mediterranean there is a general current flowing along the shore, near the western end, whose direction is from the west to the east on the coast of Africa, and from the east to the west on the coast of Europe. On the French coast the velocity of this current does not exceed 3 inches per second, whilst at Algiers it has been noticed to flow at the rate of from 10 to 12 inches, or even as much as 40 inches, per second, at the extremities of some capes.

The oceanic currents are of much greater importance than that of the Mediterranean, on account of the immense volume of water they roll along, and of their velocity; as likewise of their greater influence upon the climate of many portions of the globe, and the serious interference they exercise upon the operations of sailors. The Atlantic currents are those with respect to which we possess the most accurate information; and, indeed, it is probable that Major Rennell's surmise may be correct, that the outline of the shores of the Pacific is not so calculated to give rise to them as those of the Atlantic. The most important of the currents in the

latter is the great Gulf stream, which sets round the Cape of Good Hope from the Pacific, along the western shore of Africa, until it meets the Bight of Benin, by which it is deflected to the opposite coast of America. Striking the extreme eastern point of that continent at Cape St. Roque, it continues along the American shore to the head of the Gulf of Mexico, from which it flows through the Straits of Florida, following the eastern shore of the northern continent for a short distance, and a little below Newfoundland it turns off abruptly to the eastward. The main body of the Gulf stream is then deflected towards the south, and is lost near the Azores; but an important branch sets towards the direct east, and runs round Ireland and England, finally losing itself in the Arctic Ocean, near Spitzbergen. The effect of this stream is decidedly to contribute to the mildness and moisture of the climate naturally arising from our insular position.

On the western coast of France a current takes its rise, which flows in a southerly direction along the coast of Africa, having, in its course, given off the stream before noticed as flowing into the Mediterranean: it is finally lost in the Bight of Biaffra. Major Rennell was of opinion that a similar current took its origin in the Bay of Biscay, and flowed in a north-easterly direction round the British Islands. It is, however, doubtful whether this be not merely the branch of the great Gulf stream already noticed.

The progression of the tide-wave creates a current, whose direction, when observed near the shore, is very variable; although, as a general rule, it will be found to alternate, like the cause which gives rise to it. But it would appear that the change in the horizontal direction of the current by no means coincides invariably with the vertical change of the tide in time. There is an interval, more or less long, between the reversal of the direction of the current and the epochs of the full or the low tide; and the coincidence,

when it does occur, appears to be owing to some local and exceptional circumstance. Again, in some positions the direction of the tidal current varies through nearly all the points of the compass within the day (as, for instance, in the channel between Jersey, Guernsey, and the French coast); in other cases the variation only extends through a portion of the circle; whilst the normal change is only in the precise direction of the ebb and the flow.

As the principal lines of the tidal currents thus coincide with that of the progress of the great tidal wave, it must be evident that they would vary on each shore of the British Islands. On the southern shore they are principally from the south-west to the north-east, and *vice versâ;* on the western coast they are from S.S.W. to N.N.E., and *vice versâ;* on the eastern coast they are from N.N.W. to S.S.E., and *vice versâ;* which directions continue until the tidal wave which has passed round the northern part of Scotland, and continued down the eastern coast, meets the tide wave which has passed along the southern shore. But the influence of the configuration of the land upon the directions of currents is very perceptible in the shallow seas surrounding the British Islands; and it is therefore in similar positions more important to observe the nature and effects of local currents than to adopt implicitly any merely abstract theoretical deductions. The peculiar laws of the tides already alluded to as prevailing upon the coast between Portland Bill and Selsea Point, and in the Bay of the Seine, may be cited as instances of the modifications the outline of a coast is able to produce in the direction of the tidal currents. Upon the southern coast of Great Britain another cause of irregularity exists in this, that the continuance of the south-west wind for a lengthened period will cause the tide to flow for an hour longer than usual, and in this manner give a preponderance to the direction of the current in the direction of the flood over that of the ebb.

The velocity of oceanic currents is not affected by the movement of the waves, for the advance of floating bodies within their influence takes place as rapidly when the sea is agitated as when it is calm; but the wind has very considerable power upon them, especially in their upper portions. It is also observed that occasionally there is a difference of direction between the upper and lower strata of currents, which may in all cases be accounted for by the interference of some local disturbance. The rate of flow varies within very considerable limits, as may be perceived from the following list of some of the most remarkable currents hitherto observed.

	ft.	in.		ft.	in.	
At the Ile d'Yeu, off the coast of La Vendée	1	8				per second.
At the harbour of Lorient	3	4				„
„ „ Cherbourg	4	8				„
Off the coast of Alderney, sometimes	11	6	to	14	6	„
„ south coast of England	5	0	to	10	0	„
„ entrance to the port of Hâvre	5	0				„
„ „ „ Dover	6	6				„
„ „ „ Calais	7	10				„
„ „ „ Dunkirk	4	8				„
„ coast of the Orcades Islands	13	0	to	14	8	„

The effect of the interference of any irregularity of the outline of a coast upon the direction of the currents is to produce a series of counter-currents, eddies, and whirlpools. It is admitted, therefore, that any abrupt projection or retreat from the general line, either of a current or of the bed of a river, will give rise to such counter-currents, whose destructive action upon the obstacles causing them will have the greater intensity, in proportion to the greater velocity of the stream. Capes, bays, and islands are the natural means by which these effects are produced on a large scale. The jetties at the mouths of harbours have a precisely similar action; and the changes they reciprocally produce upon

the normal direction of the main currents require to be carefully studied, both by engineers and pilots.

At the embouchures of rivers the effect of the disturbing causes is further complicated by the dynamical tendency of the water of the current to flow into the depressions always existing in such positions at low tides, and by the descent of the upland soft waters. The form of the embouchure has considerable influence upon the phenomena to be met with in any particular instance; for it may happen, and sometime does, that even when the tide falls the sea water will continue to be poured into the river; or *vice versâ*, that the river will discharge into the sea after the tide has commenced rising; inasmuch as the conditions of the interchange of the waters depend upon the configuration and size of the passage. Thus, at the mouth of the Adour, a river which, instead of being open towards the sea, has a funnel shape with the neck outwards, the tidal wave maintains a greater height in the sea than in the river during the whole of the flood, because it cannot enter through the narrow mouth with sufficient rapidity to fill the bed of the river. The difference of height has been noticed to be as much as 4 feet during the syzygies; and, in consequence of it, the duration of the flood-tide in the interior of the Adour is prolonged for about an hour after it has ceased in the open sea. Analogous effects have been observed at the mouth of the Tay, in Scotland, where also the embouchure is suddenly contracted.

A very peculiar and interesting phenomenon, before alluded to, may be observed during the rise of the tide in rivers. In such cases, as the molecules of the water pouring into the river are impressed with a velocity equal to that of the great tidal wave, their direction whilst flowing into the river will evidently be a resultant from the two directions of their original flow, and that arising from the tendency of the water to fill up the depression of the embouchure. In the centre of the principal current there will then be formed a line

corresponding with the axis of equilibrium of the forces attracting the water up the river and towards the side. This remarkable line is sometimes sufficiently defined to be visible to the naked eye, and at all times it may be distinguished by a broad sheet of stagnant water on either side; but it necessarily changes, both in its direction and its form, according to the state of the tide, or the nature of the bed of the river. A precisely similar effect we have seen to take place during the rise of the tide in certain bays, as in the cases before cited of the Bay of Weymouth and of the Seine; but its mode of action may be more distinctly observed in rivers.

The descending fresh water interferes with the transmission of the tidal wave up a river in the following manner:—As soon as the flood begins to pour in it creates a species of dam, opposing the descent of fresh water. Until the sea has attained a greater height than that of the water so accumulated it cannot flow into the river; but as the rise of the tide is usually much more rapid than that of the land-waters, its effect becomes quickly perceptible. In the meantime a series of zones of still water, of eddies, currents, and counter-currents will be formed, in consequence of the opposite directions of the flow of the two waters and of the difference in their specific gravities; and these irregularities will assume a greater importance, and a more permanent character, in proportion as the volume and the velocity of the descending fresh water is more considerable; and they will continue until the tidal wave shall have entirely overcome the resistance of the downward current of the river.

It is on account of the interference of the descending fresh water with the transmission of the tidal wave, that the hours of the flood and the ebb tides are retarded in proportion as the distance from the sea increases; at the same time, however, the species of heaping up produced in them augments the difference of level. Indeed, from this cause it

frequently happens that in the interior the tide rises to a point considerably above the level of the high tides in the ocean.

But the most singular phenomenon connected with the rise of the tide in rivers is one presented by the "bore." According to Colonel Emy, this may be defined as being a peculiar undulation, which announces the arrival of the flood-tide in many rivers. It consists of two, three, or sometimes four waves, very short, and succeeding one another rapidly, which bar the whole river, and ascend it to a great distance; they often break upon the crown and upset everything they meet in their course, and are accompanied by a fearful noise. In the Severn, the bore is stated to be of almost daily occurrence, and sometimes even to attain a height of 9 feet; in the Dordogne it rises from 5 to 6 feet, and travels at the rate of about 5 miles in 34 minutes; in the Seine it does not exceed 3 feet; in the Thames it only exists in a rudimentary state; whilst in the Hoogly, at Calcutta, it rises about 5 feet, and is transmitted at the rate of about 17½ miles per hour; and in the Menga the rise is said to be 12 feet.

The cause of the bore is universally considered to be owing to the interference with the transmission of the tidal wave, arising either from the sudden contraction of the embouchure of the river, or from the existence of some abrupt step or bar in the bed. The wave terminates abruptly on the inland side, because the quantity of water contained in it is so great, and its motion so rapid, that there is not sufficient time for the surface of the river to be raised immediately by the transmitted pressure. Dr. Whewell compares this abrupt tide-wave to those which curl over and break upon a shelving shore. The periods at which its effects are the greatest are at the syzigies; and they decrease in proportion as the bed of the river is deeper.

EFFECT OF WAVES, TIDES, AND CURRENTS UPON SEA COASTS.

The sea-shore, of whatever materials it be composed, if it lie in the direction of any current, whether tidal or of any other description, is gradually worn away by the incessant action of the water. Under normal circumstances there is a singular uniformity in the mode of this degradation; and equally in the cases of hard rock, of agglutinated shingle, or of clay, it will be found that, for a certain height above the level of ordinary calm high tides, the outline of the shore assumes a curvilinear form. Where the sea is much agitated, the height in question may attain from 13 to 14 feet; the curve itself is always cycloidal. At its foot, and tangentially to it, succeeds a slope, which joins the natural beds at the level of the lowest tides, with an inclination varying according to the nature of the beach. Sometimes the slope within the range of the tides is as 7 to 1; and it diminishes as frequently below the level of the low-water mark to as much as 80 to 1. At other times, and especially if the bed be of mud, the slope becomes almost abrupt below the low-water line, because the water supports the mud. At Cherbourg, it was noticed that the small materials thrown into the sea for the formation of the breakwater took a slope of about 45° below that line; and in many cases upon our shores the shingle banks may be observed to assume a similar inclination. In the Lake of Geneva, the shore near Vevay, where it is composed of fine sand, takes a slope of 10 to 1, to a depth of from 6 to 7 feet below the variations of the water-line; whilst, at a greater depth, the slope is as 2 to 1, or the natural inclination of sand and still water.

The destructive action of the sea upon the shores bounding it arises principally from the action of the waves moving in the direction of the prevailing wind. This action is complicated in its nature, but it may be considered to be composed of the oscillating motion of the molecules of the water

occasioned by the waves; of the effect produced by the wind upon the upper part of the waves themselves; of the reaction produced by any projection beyond the ordinary line of the shore; of the permanent and periodical currents to which the mass of the water may be exposed; and, finally, of the dynamical effort exercised by the water set in motion. Of these causes, the three first are without comparison the most powerful, and hitherto it has not been ascertained within what limits they are able to act. But it is important to observe, that the existence of any object in abrupt relief gives rise to such eddies that the bottom of the sea round it will be rapidly carried away, if it be of a nature to yield easily, and be situated within the limits of the disturbing causes. It follows from this, that great precautions must be taken in the construction of any vertical retaining wall whose foundations may be near the low-tide line; for the repercussion of the waves is certain to undermine them, if formed in clay or light sand.

Occasionally it may happen that the destructive action of the sea, which principally operates in the direction of the advance of the flood-tide, is reversed by the existence of some local current which increases the power of the ebb. Thus in the British Channel the outline of the majority of the bays between Land's End and Portland Bill is concave towards the incoming tide; but between Portland Bill and Selsea Point the outline of the bays, of the mainland at least, is almost invariably convex to this tide. The cause of this apparent anomaly is to be found in the existence of the second tide before noticed, which increases the volume of the ebb close in shore; and also to the fact that the island of Portland, and the spur terminated by St. Alban's Head, form, as it were, breakwaters sheltering the intermediate district from the effects of the south-west winds.

The geological nature of the rocks exposed to the fury of the waves may also materially affect their rate of destruction,

and also re-act upon the preservation of the neighbouring shores. In the particular instance last cited a remarkable example of this may be found; for the projecting points of the Isle of Portland and St. Alban's Head are respectively of the harder and more resisting strata of the oolite; whilst the intermediate bays of Weymouth and of Poole are excavated in the less resisting strata of the Oxford or the London clays.

The materials detached from the rocks or shores by the sea are transported by the currents in a direction which may be stated generally to correspond with that of their greatest and most permanent influence, whether that be in the same line as the advance of the great tide-wave or in the opposite one.

The shingle, or other alluvions, which surrouud the bases of cliffs, follow precisely the contours and directions of the latter in plan. But when the shingle collected in a bay has no extrinsic support, it will assume a curvilinear outline, whose concavity will be turned towards the sea in the direction of the prevailing wind. This effect may be distinctly perceived in some of the bays upon the eastern shores of England, and upon the French coast between the Caps d'Antifer and Grisnez; for in both cases the yielding nature of the materials allows the general outline of the coast to modify itself to the most powerful action of the waves, which, as is well known, is always exercised in the direction of the prevailing winds.

If, during their advance along the shore under the double impulsion of the waves and of the currents, the alluvions meet a deep bay sheltered from the prevailing winds, the still water existing in it will allow the heavier particles to subside; and if, in addition to the general comparative tranquillity of the bay, there should exist any subsidiary currents, producing upon their edges zones of still water, the remaining matters in suspension will be deposited either tem-

porarily or permanently. This latter contingency occurs in the embouchures of most rivers, and the bars are formed, in such cases, by the deposition of the alluvions upon the line of junction of the littoral current with that of the down stream; and, as these directions are usually either at right angles, or highly inclined, to one another, the axis of the bar follows that of the resultant affected by the volume, the velocity, and the direction of the two streams.

Should the alluvions, instead of meeting with a deep bay, such as has been described, meet with a projection upon the shore, they will form a deposit in the opposing angle which will be concave to the general direction of their advance. If the action giving rise to the alluvions be permanent, it will carry them round the projection, and there, meeting with still water, they will be deposited. The accumulation of a bank in this position will often be accelerated by the currents and counter-currents generated by the projection.

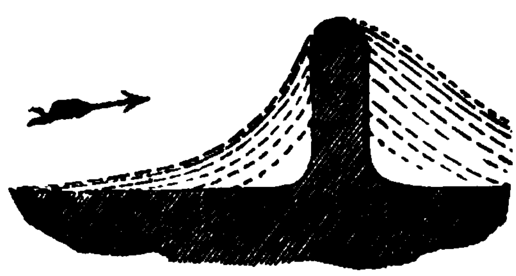


Fig. 181.
Projection in a Current.

Some very striking illustrations of the importance of the effects produced by the alluvions set in motion by the sea, whether they consist of shingle, sand, or silt, may be found upon almost every shore. On the coast of Kent, two deep indentations upon the outline of the land, at Romney Marsh and Pegwell Bay, have been filled in. On the coast of Norfolk, Lowestoff Ness is also gradually gaining upon the sea, from the deposition caused by an interference with the progress of the shingle. In the Mediterranean, the ports of Aigues, Mortes, and of Fréjus, from which the armies of St. Louis embarked, are now far inland, and are only kept open for small vessels by means of constant dredging. The port of Ostium, at the mouth of the Tiber, formed by Claudius and repaired by Trajanus, is now about three miles from the sea. The lagunes of Venice tend naturally to silt up, and

are only kept open by means of the artificial channels constructed for the purpose of carrying the waters of the several rivers Brenta, Bachelone, Piave, and the Sile, more directly into the open sea of the Adriatic. Illustrations of the erosions produced by the action of the waves may also be found on every shore.

The materials detached from the cliffs undermined in this manner are rolled forward continually, and under the influence of this friction they become comminuted at length to a fine sand. In some cases the sand, earth, and broken shells deposited by the high spring tides are carried inland by the wind, and advance in waves nearly as strongly defined as those upon water. If any obstacle be offered to their protress they accumulate to a vast height, and occasionally they spread over a great extent of land on either side. Thus, in the Landes of Bordeaux, hillocks of sand are said to have been formed attaining a height of about 160 feet; and the sands advanced towards the interior at the rate of about 88 feet per annum, until Brémontier commenced the series of works for the purpose of fixing them which has contributed so much to immortalise his name. In the case of the Landes of Bordeaux, the progress of the sand was accelerated by the dryness of the atmosphere; but upon our own coasts in Poole Harbour, and on the French coasts near Dunkirk, the rain-fall, and perhaps also the presence of a considerable quantity of earth in conjunction with the silicious sands, develops a peculiar vegetation which prevents the further progress of these downs, or dunes, as they are called in France and Belgium. The principal measures adopted by Brémontier consisted in planting the sand reed (*Arundo arenaria*) for a distance of about 300 yards from the sea. For a second zone, of about 1,000 feet in width, he planted creeping plants, brambles, heaths, &c.; and more inland, at a distance beyond the influence of the salt water, he planted a zone of fir-trees. It appears, however, that the

fixing the landes near the Plains of Soulac and Thalais, on the sea-shore to the south of the Gironde, has produced a considerable effect in the action of the sea upon the outline of the coast. The very incoherence of the sands appears to have prevented their removal, seawards at least.

CHAPTER III.

SEA DEFENCES—GROINS.

The foregoing general observations upon the action of the sea upon the coasts, or works exposed to its effects, are necessary to a complete knowledge of the most advisable means to be adopted for their protection. With respect to the defence of coasts, it follows, from what has been said, that the works to be executed must consist either of such as are able to break the force of the waves before they reach the shore; or of such as are able to consolidate the shore itself, so as to enable it to resist more effectually the denudation produced by the waves; or of such means as shall cause an accumulation of sand and shingle upon the fore shore. Under some circumstances it may be advisable to combine the three descriptions of work.

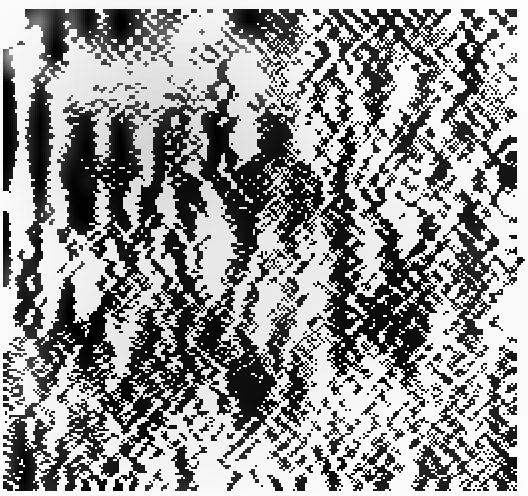


Fig. 182.—Groin.

The construction of groins firmly connected with the shore, and following a direction normal to that of the reigning winds in tempestuous weather, whether they be submersible or not, would diminish the action of the waves; particularly if they be carried out so far as to prevent the species of cascade, which always takes place on their down side, from

affecting the shore itself. In some cases isolated pillars of masonry or timber-framing, placed like the squares of a chess-board, by causing the waves to break seawards, may become very efficient means of defence. On the coast of Holland very successful results have been obtained by the erection of wooden framing placed in a parallel direction to that of the coast at the line of low water. In soils susceptible of being easily removed, however, it is indispensable to protect the foundations of these groins by apron pieces.

[One of the most ancient kinds of defence for sea-shores is the system of groins or jetties run out from the shore, for the purpose of arresting the travelling shingle, the collection of which forms a natural barrier against the waves. The common form of groin is seen on the shores of Kent and Sussex, consisting of a row of piles from 6 feet to 10 feet apart, held back by land-ties and piles on the weather side, and backed by longitudinal planking, against which the beach is collected. The groins are usually laid in a direction at right angles to the shore; though they are in some cases placed at an acute angle or an obtuse angle to the waves.

The motion of shingle on any beach is dependent upon the extent to which it is exposed to the waves, the angle at which the waves break upon it, and the violence of the waves. The disposition of groins, then, should be so adapted as to give the prevailing waves the least favourable means of forcing the shingle forwards, and to assist, as much as possible, the less violent and constant winds, in driving it back again. A direction slanting from the prevailing winds appears to be the most favourable. If groins be made too lofty, over-falls of water are created, and the shore is gullied out on the leeward side. An important application of groins, illustrating the excellent effect of their employment in arresting the encroachment of the sea, and aiding

in the reclamation of the foreshore, is supplied in the works for the construction of the Sunderland Docks, for which Mr. John Murray was the engineer.* Mr. Wm. Brown, writing in 1849, stated that the sea had been gradually, but somewhat rapidly, encroaching on the Town Moor, Sunderland, and upon the ground occupied as a fort, the height of this table-land about the high-water mark varying from 32 feet to 36 feet. The banks were composed of a hard marly clay, standing nearly vertical, presenting the appearance of a cliff, resting on limestone, with a slight accumulation of sand and gravel at the bottom. Between the years 1737 and 1845, a period of 108 years, an average breadth of 143 feet had been carried away. At a point opposite the corner of the barrack wall an inroad nearly 300 feet in breadth had been effected. To aid in forming a more extended beach, on which the sea might expend itself, and be prevented from reaching the foot of the banks, two timber groins were constructed, composed of whole timbers having their lower ends let into the rock, and their upper ends secured to raking braces, and the face sheathed with 6-inch planks. They fully answered the purpose of protecting the moor; and an area of about two acres of land was permanently formed at the foot of the cliff, consisting of sand and gravel.

An independent system of groins was subsequently laid down, to co-operate in making land for the construction of a new dock and other works to the south of the river, and within, or to the seaward of, the line of high-water mark. The groins were erected for the purpose of retaining the material deposited on the foreshore, and of arresting the sand and shingle, which naturally travelled to the southward in order to form a barrier-beach, which should effectually exclude the sea from beyond a given line.

* *Proceedings of the Institution of Civil Engineers*, 1849, vol. viii. p. 186; vol. xv. p. 418.

Seven groins were constructed, extending over a frontage of about 2,500 feet in extent,and at intervals of from 350 feet to 450 feet apart. Of the first, second, and third groins, the inner ends stand 10 feet above high-water line of ordinary spring-tides, and about 20 feet above the surface of the limestone rock; the seaward ends are only 2½ feet above the surface of the rock. They are respectively 326 feet, 326 feet, and 358 feet in length, and are formed to an inclination of 1 in 18. The north side is inclined with a batter of 2½ inches in one foot, and the south side with a batter of 1 to 1. The top, or crest, is formed as an arch to a radius of 5½ feet, with a bonding course 15 inches thick, at a distance of 9 feet below the crest. In the interior, there is a backing of coursed rubble at the north side, making up a thickness of 6 feet at the bottom and 4 feet at the top. Above the bonding course, the space is filled with coursed rubble, well packed and built with mortar; below, the cavity is filled with the excavated magnesian limestone.

The set of the flood-tide in the offing is S. ½ E., and at spring-tides it runs with a velocity of 3½ miles per hour. Along the shore, the set of the tide was S.S.E.; and, owing to a counter-current, the first flood, as well as the time of high water, was about two hours earlier, and, at the distance of half a mile, about one hour earlier than in the offing. This flood-tide was first caught by the north pier, which acted as a groin, where a considerable portion of the land brought up by the tide was deposited. In fact, upwards of 35 acres of land have been recovered from the sea by the erection of this pier. The sand settled at a slope of from 1 in 40 at the upper end of the beach, to 1 in 60 at the low-water mark. The quantity of sand retained was not constant, for the large amount accumulated during the neap tides and in calm weather was much lowered by a heavy sea: as much as 5 feet in vertical height has been carried away in a single tide. A portion of the dislodged material

was caught by the south pier, and was carried up the river until it arrived at the Potato Garth on one side of the river, or at the Wave and Sand Trap, or Beaching Basin, on the other side. In these two places a portion of the sand was deposited, and the remainder continued its course up the river until it was checked by the land-water, by which it was caused to subside and form sand-banks, which were washed to the sea by the descending freshes. But the quantity which passed up the river is comparatively small; for the south pier is shorter than, and is overlapped by, the north pier, and the largest portion travelled onwards until it was arrested by the groins, which, together with the excavated material deposited between them, formed a foreshore, or barrier beach. The inclination of this foreshore was about 1 in 25, whilst at the bottom of the banks to the southward, where there were no groins, the inclination was 1 in 11, and sometimes even steeper. The flood-tide, after it passed the south pier head, proceeded directly to groin No. 1, by which it was turned, so as to form an eddy or counter-current, returning along the beach northward, and the glacis of the pier eastward, and rejoining the main set of the tide, in completing a circuit of 1,200 feet.

In consequence of the eddy just described, and perhaps also of the form of the groin No. 1, the surface of the sand on its north side was 2 feet lower than on its south side; whereas, at each of the other groins, the sand at its north side was fully 12 inches higher than at the south side. It was judged that by the form of the groin, with the southern slope of 1 to 1, it acted as a conductor to the waves which struck it, and to carry them forward so as to scoop out the sand that had been accumulated during calm weather.

Another experience was derived, at the time of the equinoctial tides, during the spring of 1848, when the groins were completely isolated. An unusually heavy sea made a breach in No. 1 and No. 3 groins, at the same time loosen-

ing some of the stones in No. 2. All these failures occurred at one level—the intersection of the crests of the groins with the line of high water of ordinary spring tides, indicating that the waves acquired their greatest power at high water. It was observed, in fact, that at an interval of 1¾ hours before high water, the velocity of a wave at the South Pier head was 9 feet per second, and its height 5 feet 9 inches; whereas, at high water, the velocity attained a maximum of 13⅓ feet per second, with a height of 6 feet. At an interval of 1¾ hours after the turn of the tide, the velocity of the waves meeting the descending water was only 7¾ feet per second, though they were 7⅓ feet high.

In view of the circumstances above detailed, the remaining groins, Nos. 4, 5, 6, and 7, were constructed to semicycloidal contours. Nos. 4 and 5 groins were designed to have lengths of 442 feet and 510 feet respectively, with inclinations of 1 in 26 and 1 in 30, so as to stand 10 feet above high water at the landward ends, and 7 feet below seaward. They were actually constructed to lengths of only 312 feet and 360 feet, as they were met in the course of construction by the deposited material, consisting of the hard blue clay which forms the banks. The sides were brought up of ashlar work. The hearting was of large-size rubble closely packed, the vacancies between it and the ashlar being filled with small stones and Roman cement. At the depth of 6 feet below the crest, ashlar masonry was laid on the rubble and carried up near to the crest, leaving a small vacancy which was filled with small rubble and Roman cement. The groins were additionally strengthened for a distance of 80 feet landwards of the high-water intersection by ashlar introduced at the depth of 8 feet below the crest.

Groin No. 6 was laid at an inclination of 1 in 32; and No. 7 at 1 in 30. No. 6 is stopped short at a level 2½ feet above high-water intersection; No. 7 stops short at that intersection.

The groins of semi-cycloidal form stood well. This peculiar form was adopted for the purpose of better binding the masonry, by the spiral line formed by the joints of the diminishing arch.

The efficacy of groins has, under other circumstances, been signally manifested on the east coast, north of the Humber. The coast has been wasted in various ways, according to the material and its configuration. Opposite Hilston Church and some other places, for instance, the cliffs are about 60 feet high, having a stratum of from 15 to 20 feet of boulder clay at the base. The waste is not by any means regular, but occurs at intervals in huge landslips, from 200 to 600 yards long, and from 20 to 50 yards in width. In time, the waves undermine the foot, and by the aid of frost and land-water bring down fresh slips. The loss is at the rate of from 1 yard to 1½ yards in width per year. Two such landslips are represented in Figs. 183 and 184. The base, A, of boulder clay, stands up as a revetment wall, almost vertical, for 20 feet in height. B is the old drift, of sand, gravel, clay, &c.; at C is a fresh-water deposit about 5 inches thick, from 60 feet to 65 feet above the sea-level; D is the "new drift," or surface covering. One of the slips, where the cliff is 70 feet high, is shown in an early stage in Fig. 185, when the sea, after having cleared away all traces of the preceding slip, acts at the foot of the base of boulder clay, undermining it until the superincumbent earth eventually breaks away, and slides gradually down to the foot. Slips of this kind occur where the beach is unusually low, and admits of deep water at the foot of the cliff.

For many years it had been the practice to remove the shingle, or debris of the cliffs, for sale for the repair of parish roads. Opposite Withernsea, it appears, from 200,000 to 250,000 tons had been removed along a frontage of two miles, between the years 1854 and 1869, being at the rate of 8,000 tons per mile per year. It is remarkable that,

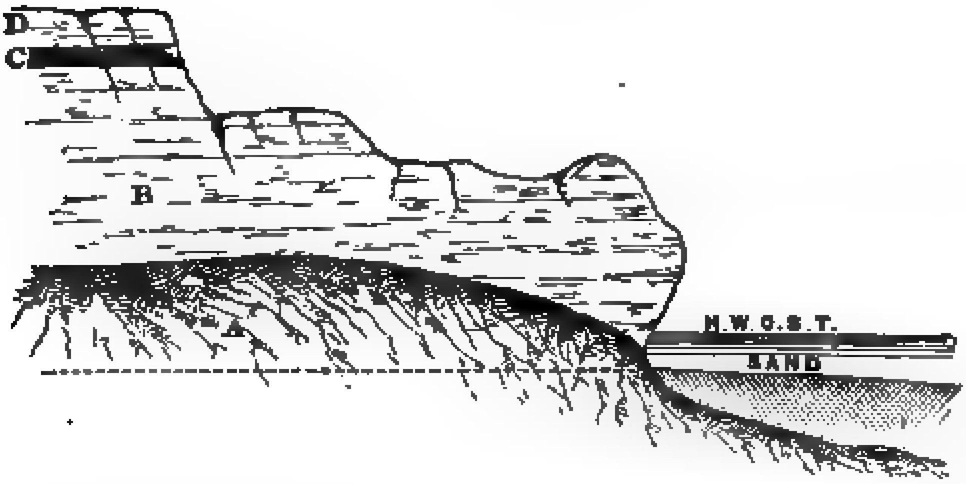


Fig. 183.—East Coast: Landslip.

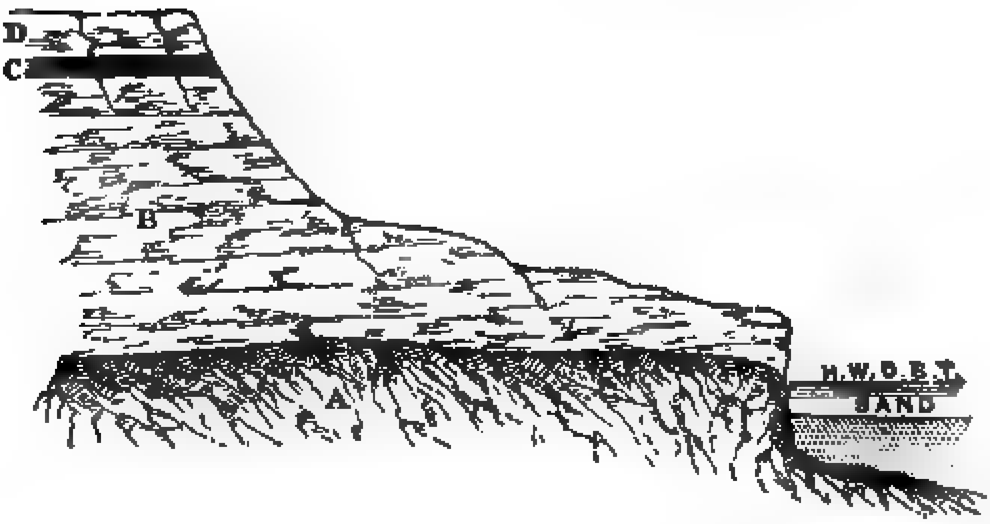


Fig. 184.—Landslip.

Fig. 185.—Landslip.

whilst, prior to the gravel trade, the waste of the cliffs only varied from 2·10 feet to 4·20 feet per year, the waste was increased from 9 feet to 18 feet per year after the removal of the shingle was commenced. In 1870-71,* six groins were erected, as shown in Fig. 186, on the foreshore, which was very low; and, after a north-west gale, the

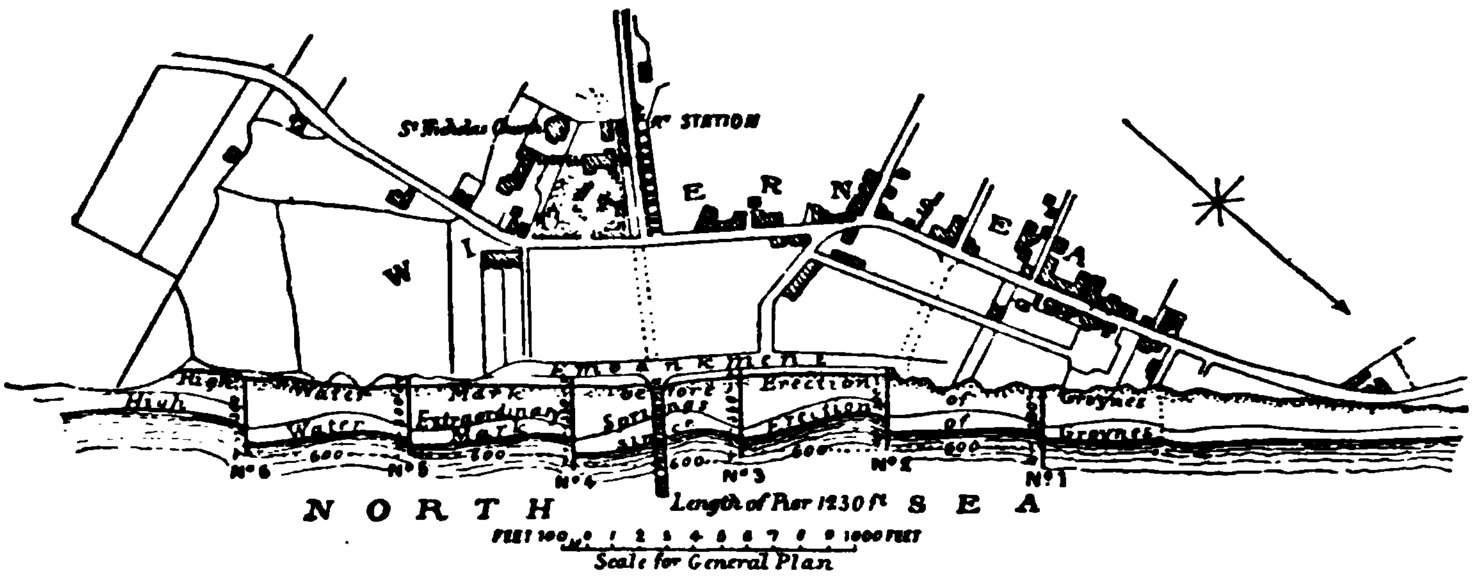


Fig. 186.—Groins, Withernsea, East Coast.

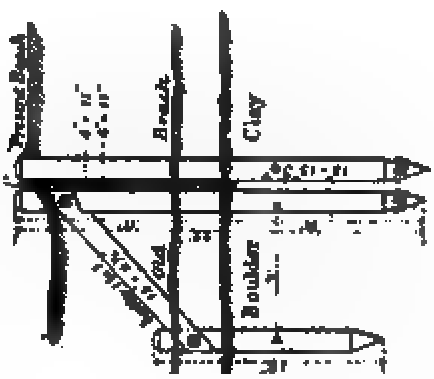
whole of the beach, occasionally, was swept away down to the bare clays. On the 8th February, 1868, an unusually high tide occurred, doing alarming damage along the coast. The high-water mark of this tide reached a point 400 feet to the landward of the present ordinary high-water mark. The groins, constructed of Dantzic redwood, were put down at intervals of 200 yards, at right angles to the beach; and they varied in length from 300 to 350 feet. No. 2 groin is represented in elevation and plan, Figs. 187. The tops of the groins at the land end are 12 feet above ordinary high-water spring-tides; the outer ends are from 3 to 4 feet above the beach. The groins, shown in detail, Figs. 188, were strutted at the south side to enable them to resist the pressure of the accumulated beach at the north side. The

* See Mr. R. Pickwell's paper on "The Encroachments of the Sea from Spurn Point to Flamborough Head, and the Works executed to Prevent the Loss of Land," in the *Proceedings of the Institution of Civil Engineers*, 1877-78, vol. li. p. 191.

main piles are 18 inches square, 22 to 24 feet long, and shod with 14-lb. shoes of wrought iron. These piles were driven 11 or 12 feet into the boulder clay below the sand. The strut piles, 18 inches square and 12 feet long, were driven 8 or 9 feet into the clay. The struts, 18 inches wide by

ELEVATION & PLAN OF GROYNE No 2
Showing Beach Surface before Erection and since

Figs. 187.—Groins.



Figs. 188.—Groins.

6½ inches thick, are dovetailed and halved on to the piles, and fixed with 1¼-inch bolts and nuts and cast-iron washers. The planking is 4 inches thick and 11 inches wide, in lengths of from 20 feet to 25 feet, fixed to the piles with 1-inch bolts and nuts. It had been observed that the breakers were most destructive about the locality of high-water mark, or during 1½ hours before and after the time of high water,

and here 30-feet piles were driven. The cost of the groins was £3 7s. 6d. per lineal yard.

Since 1871 no more land has been lost along the coast thus treated. On the contrary, in June, 1876, five or six years after the groins had been erected, Nos. 3, 4, 5, and 6 were found to be completely buried under the accumulated shingle and sand, and Nos. 1 and 2 were buried for two-thirds of their length, according to Figs. 187. The ordinary high-water spring-tide mark is from 50 to 80 yards further seaward now than formerly. The protective efficiency of the groins is only apparent for a length of about a quarter of a mile to the southward of the last groin, where the sea at high water still reaches the cliffs. But to the northward the efficiency of the groins in preserving the cliffs from the action of the sea at high water extends for upwards of a mile. Mr. Pickwell believes that the groins might have been, with equal benefit, placed at intervals of 300 yards instead of 200 yards.

Mr. Pickwell, referring to the difficulty of penetrating the wet sand beach down to the clay, in order to fix the lower planks of the groins just described, proposes a combination of sheet piles at the lower part, and longitudinal planks above. A groin on this arrangement would be more easily constructed than the groins shown in Figs. 188; it would also be stronger, besides offering a minimum resistance to the waves, if the upper planking be gradually added as the beach accumulates, maintained at a level of 3 or 4 feet above the beach. The cost is estimated at £4 10s. per lineal yard.]

CHAPTER IV.

SEA DEFENCES—DYKES.

The consolidation of a shore must be performed in very different manners under different circumstances. If the natural inclination above low-water mark, up to the extreme point reached by the waves during tempestuous weather, be abrupt, it will probably be found more advantageous to interpose a vertical wall or defence, whether of masonry or of woodwork. In the former case the stones should be laid as headers, and the upper parts coped with the largest stones it is possible to procure, also laid as headers; and it would be advisable to pave for a distance of some 8 or 10 feet beyond the coping, so as to throw off effectually any water breaking over the wall. The footings must also be protected by rubble stone-work, or by an apron bedded in mortar.

In Holland, however, the dykes are very frequently formed as represented in the sketches here given, the body of the embankment being of earth with a hearting of fascines, or of bundles of reeds in some cases, or with a facing of similar materials in others, protected at the foot by rubble-stones. Or, again, the whole of the embankment may be in earthwork, with or without any provision to break the force of the waves. Many instances may be mentioned where the banks of the polders are formed in a manner similar to a coffer dam, by means of a puddle-dyke retained between sheet piling. At Hâvre, the mode which proved most successful consisted in forming an embankment of earthwork behind a

vertical inclosure of sheet piling, with a loose rubble apron at the foot, or even, in the positions where the scouring effects of the current were very great, with an apron of solid masonry. The general form of the sea defences of the plains near Hâvre is represented in the sketch Fig. 195.

Fig. 189.—Dyke.

Fig. 190.—Dyke.

Fig. 191.—Dyke.

[Dykes, or sea and river embankments, are continuous deposits of regular masses of earth at places where it is necessary to resist the pressure or the penetration of water, well united with soil of the foundation, and carried above the

level of the highest floods to which they may be exposed.

Fig. 192.—Puddle Dyke.

Fig. 193.—Puddle Dyke.

Fig. 194.—Puddle Dyke.

Fig. 195.—Puddle Dyke, Havre.

In the Netherlands, or lower lands, the very existence of

large districts was dependent upon the dykes constructed for the reclamation of land from the sea—a process of great antiquity. In the fourteenth century an extensive inroad of the sea was made by a severe flood, by which the greater portion of the country now occupied by the Zuyder Zee was inundated.

The dyke, Fig. 196, consists fundamentally of three elements: 1st, the crown, or central portion, the rectangular prism A B C D, the level of which is at least 20 inches above that reached by the highest flood or storm tides. The width, A B, varies according to the pressure to be resisted, so that the water may neither pass over nor penetrate. 2nd, the outer slope, A C E, seawards, of a length and inclination adapted to the pressure and the action of the waves or the currents. The point E is the toe of the dyke. 3rd, the inner slope, B D F, serving principally as a support for the crown or body of the dyke. The inclination is such as the earth naturally rests at, or the angle of repose. The point F is the heel of the dyke. The total surface, E C D F, is the stool, or foundation, of the dyke. A complete sea-dyke has, further, an additional base, or berm, E H, outside, at least 30 feet long. The level at the extremity, H, is that of average high-water mark for spring-floods, and the berm rises with a slope of 1 in 20. It aids in carrying off rain and sea-water, and for turning the waves from the dyke. Also,

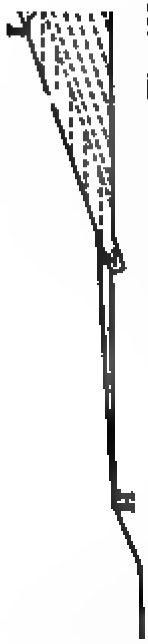


Fig. 196.—Dyke, in the Netherlands.

an additional base, or berm, F G, from 18 feet to 24 feet wide, at the height of ordinary floods of inland streams,

having a fall of 1 in 20, to carry off rain-water. The inner berm serves as a local road, and for carrying materials for repairs of the dyke. But the inner berm is usually separated from the country by a boundary ditch, G, known as the berm ditch, for the security of the dyke, and to receive the drainage from the dyke.

The most durable and least costly covering for dykes is the natural grass sod, which when well thickened, efficiently protects the dyke against the entry of water. In proceeding to the construction of a dyke, the stool or seat is cleared of stone and other rubbish; and, if it is covered with grass, the sods should be taken up and stalked for use in clothing the dyke. The ground is turned over a few inches deep with the spade or with the plough, and well broken up, that the new soil may unite well with the old soil. A ditch, a foot deep, is rabbetted into the ground at each side, to form abutments for the heel and the toe of the dyke. The material should be deposited in thin layers, 10 or 12 inches thick, if from wheelbarrows or trucks; or 14 or 16 inches thick if from horse-carts. The trampling and wheeling help to consolidate the earth. The earth is carefully broken up and well rammed; and, if dry, it should be well watered. The layers, *a c*, are arranged as indicated in Fig. 196, with an outer coating.*

The dykes on the coast of Denmark have existed for at least seven centuries. The comparatively modern dykes—constructed since, and with reference to, the great storm-flood of 1825—are raised to various heights, according to local circumstances. The highest known flood-level is taken as a point of departure, to which the height of the wave is added. The height of the outer ground, called "watt"—the space between high water and low water—has a considerable influence on the height of the waves; and, for the

* See "The Engineering of Holland," by Hyde Clark, in *Weale's Quarterly Papers*, vol. ii. 1844.

Fig. 197.—Dykes on the Coast of Denmark.

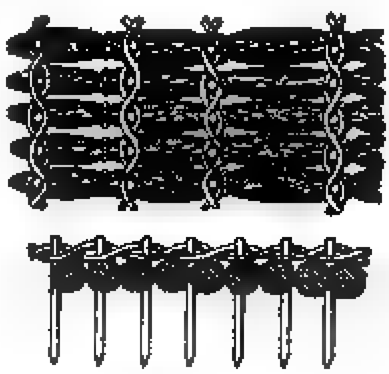
district of Schleswig, heights of from 14 feet to 18 feet are recommended. The sea slope of the dyke, Fig. 197, is not uniform, but generally a slope of 3 to 1 is adopted from the crown to a level of 10 or 12 feet above ordinary flood, completed with a "cess," or slope, of 8, 10, 12, or 15 to 1, according to circumstances. The top is from 8 to 10 feet wide in the smaller dykes, and from 10 to 20 feet wide in the larger dykes. The crests are used as roads. The inner slopes are at the rate of 1½ to 1. The best material for dykes is clay mixed with sand; and as the marsh ground itself is usually of this nature, it is generally used. Clay alone, with any sand in mixture, easily bursts after exposure to rains, or dries suddenly and cracks by excessive heat. Common soil is frequently used, but it is not so good for the purpose as sandy clay, as through passages are more easily made in soil by moles, rats, and foxes, which at high water may prove dangerous. When the most suitable material cannot be obtained in sufficient quantity for the formation of the dyke, it is customary to cover the water-slope with clay several feet in thickness. When the material is of fine sand, or of bog earth, both slopes are covered with clay; and, in some cases, it is necessary to construct a bedding of clay laid a considerable depth in the ground, as the water may otherwise find its way through the ground, and issue behind the dyke. The slopes are covered with grass sods, 6 inches

thick and 1 foot square. When ordinary tides reach the foot of the dyke and the outer ground is low, the sea-slope is treated in various ways. It may be pitched with stone blocks weighing 300 lbs. at least, having their smaller sides uppermost. They are bedded on a layer of clay and small stones mixed, 12 inches thick. The slopes are made relatively steep, having inclinations of 2 or 3 to 1, and the pitching is carried to a height of from 12 to 16 feet above ordinary flood. On solid ground this protection is almost imperishable.

Where stones are scarce the lower end only is pitched, and is laid to a quick curved slope, the body of the slope being flat; as, for instance, 15 to 1.

Straw, of rye or of wheat, in a layer 2 or 3 inches in thickness, is used for the protection of slopes when sods are scarce. The straw is fastened to the earth with straw bands pinned to the earth.

Fascines, Figs. 198, also, are used on the Danish coast, consisting of brushwood—willow, by preference—bound together in bundles, 10 feet long and 1 foot thick at the middle. Several layers are placed one over the other, with intermediate layers of rough stones.



Figs. 198.—Fascine Dyke, Danish Coast.

The protection of seaweed, together with stones, has been adopted on some dykes with advantage. At the mouth of the Helder, in North Holland, banks, nearly vertical, con-

structed of seaweed and hazelwood fascines, backed with clay, have stood extremely well when exposed to very heavy seas, possessing a considerable degree of elasticity, and vibrating for some distance along the bank when struck by a wave.

In Holland the greater portion of the dykes are protected by fascines—in Dutch *ryshout*—which are derived from copses of willows, osiers, and other tall brushwood, for the growth of which the swamps and morasses are well adapted. An example of the employment of fascines for reclamation-dykes is supplied in the mode, shown in section, Fig. 199,

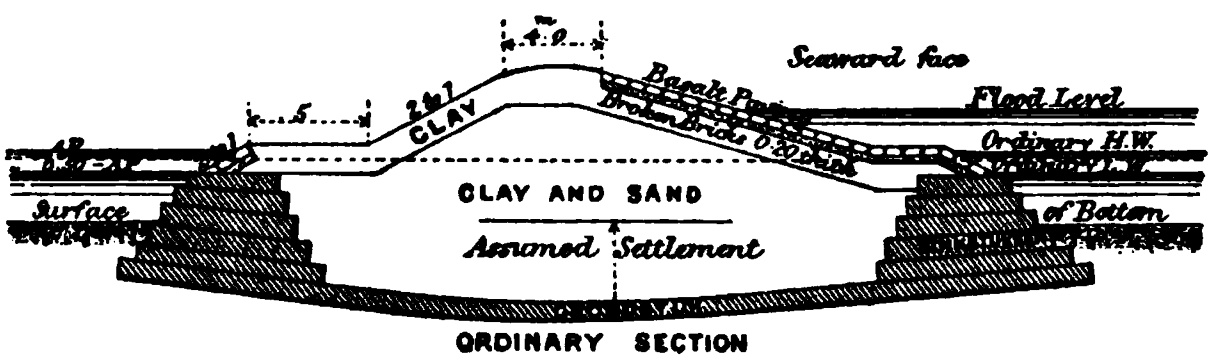


Fig. 199.—Fascine Dyke, Holland.

adopted for inclosing the estuary of the Zuyder Zee, east of Amsterdam, called "Het Y," in 1866. The width at the point selected for the construction of the dam was about ¾ mile; the depth of water varying from 5 feet to 27 feet. The bottom was soft alluvium, about 40 feet deep, reposing on sand. The rise and fall of the tide was 14 inches, but there were variations in level to the extent of from 10 to 15 feet, caused by storms, when the current was 2½ miles per hour. The crown of the dam is 13·1 feet wide, with a rise of about 10 inches, carried to 12·3 feet above A. P. (Amsterdamsche Peil, the datum of levels for the Netherlands). The sea slope is 3½ to 1 for 1·64 feet above A. P.; the inside slope is 2 to 1 to the same level. The slopes are succeeded by horizontal berms, 9·8 feet wide at the outside and 16·4 feet wide at the inside; then there are slopes of 2 to 1 for 1·64 feet below A. P. Where fascines are used, the slopes

below this level are ¾ to 1. The slopes, berms, and crown are covered with puddled clay 3·28 feet thick. Both sides were protected with strong fascine works from the top to 1·64 feet below A. P., and loaded with stone. When the dam was closed, the fascines on the inside and outside slopes were removed; a bed of broken bricks, 8 inches thick, was placed on each slope, and the surfaces were paved with stones at least 12 inches thick, to the level 1·64 feet below A. P.

The first thing done was to cover the entire site with a strong fascine mattress, worked in pieces 197 feet long, and overlapping about 3¼ feet, called "grindstukken;" then to build up the exterior portion by successive layers of fascine mattresses to low-water level, when the trough or hollow between them was nearly filled with sand or clay. The material of the core was chiefly of sand, and was securely held, none of it having been washed away by the current.]

CHAPTER V.

SEA EMBANKMENTS.

[In works exposed to the action of the sea, it is necessary to provide for and guard against the recoil of the waves, which, when driven against a vertical wall or a wall nearly vertical, fall back, and tend, by the force of their fall, to break up the works on the foreshore, and undermine the foundations. This action is the counterpart of that which takes place when the waves sweep over an embankment and are precipitated on the other side. But that action—on the foreshore—may be injuriously exerted, save upon considerably sloped work, if the slope and the protective covering be not adapted to the nature of the construction. For, admitting that the percussive action of waves, driven by the wind on sea slopes, is decreased in proportion as the impinging water is spread over an enlarged surface, the angle of the slope must be suited to the nature of the covering. Accordingly, the pitched surfaces of the Dutch dykes may be and are much more steeply sloped than the grass-covered or fascine-covered slopes.

Mr. J. R. M'Lean describes the sea embankments of the Furness Railway, constructed similarly to some of the dykes of the Netherlands. Each embankment is about one mile in length; and though the situation is generally well sheltered, the banks, during the equinoctial gales, are exposed to heavy seas. They are formed of sand, faced with 12 inches of clay puddle, into which broken stones were beaten to a depth of

4 inches, so as to form a clay concrete bed to receive the pitching or stone facing, which is 12 inches in depth. The portion of the embankment above the level of equinoctial tides was faced on each side with sods 6 inches thick, cut from the "salting." The grass, although on a slope of 1 to 1 on the inner side, is strong and luxuriant, and the parapet thus formed affords a complete shelter for the railway.

Mr. Brunlees, in designing the pitched sea embankments constructed in 1854 for the railway across the estuaries of the Kent and Leven in Morecambe Bay, made some preliminary experiments with a view to determine the inclination at which the stones ought to be placed, so as to obtain the maximum amount of adhesion between the stones, and to resist most effectively the disturbing force of sea-waves. The experimental slopes, 4 feet in vertical height, were constructed of fire-bricks, set on end, so as to equal a depth of 9 inches in the bed. The central brick in each slope was extracted by means of a weighted chain passed over pulley. The weights at the end of the chain required to extract the central bricks were as follows:—

Ratio of Slope.	Weight required to extract the Central Brick.
1 to 1	105 lbs.
2 " 1	148 "
3 " 1	144 "
4 " 1	98 "

From these results it appears that, at a slope of from 2 to 1 to 3 to 1, the maximum resistance to disturbance was presented: composed, evidently, of the resistance to gravity and the frictional resistance between the bricks. Mr. Brunlees ultimately adopted a slope of 2 to 1 for the sea-faces, at which the embankments have stood well. The sand of the bay, constituting the hearting of the banks, consists to a great degree of calcareous matter washed from the limestone

of the district. The sand stands well at a slope of 1½ to 1 in still water. The inner slopes of the embankments along the shores are at an angle of 1½ to 1; whilst those of the banks crossing the estuaries are 1¾ to 1, as in Figs. 200 and 201, showing the embankments of the Leven estuary. The

Fig. 200.—Railway Embankment, Leven Estuary.

Fig. 201.—Railway Embankment, Leven Estuary.

sea slopes were covered with a 12-inch layer of clay puddle, which protected the material during the formation of the embankment, and permanently prevented the working out of the sand through the joints of the pitching by the action of the sea. The puddle, in its turn, is protected by a bed of small rubble stones, technically called "quarry rid," which

is laid to a depth of 18 inches. On this bed, the pitching-stones, 18 inches thick at the bottom and 12 inches at the top, and averaging 15 inches in depth, are compactly set, making a total average thickness of 3 feet 9 inches of covering. The pitching, carried to a depth of 3 feet below the surface of the beach, was of limestone from the excavations and the quarries in the neighbourhood. Of the landward embankments, the land slopes were protected during the period of construction by a thin layer of puddle or sods. Of the through embankments, both sides of which are exposed to the sea until reclamation is effected, the land slopes were protected with random pitching 8 inches deep. The embankments are from 15 to 25 feet high. There is no littoral current, and sand has accumulated at the base of the bank. Besides, there are groins of rubble stone.*

Mr. G. W. Hemans adopted the same slopes for a railway embankment, Fig. 202, across the Clontarf Estuary, on the

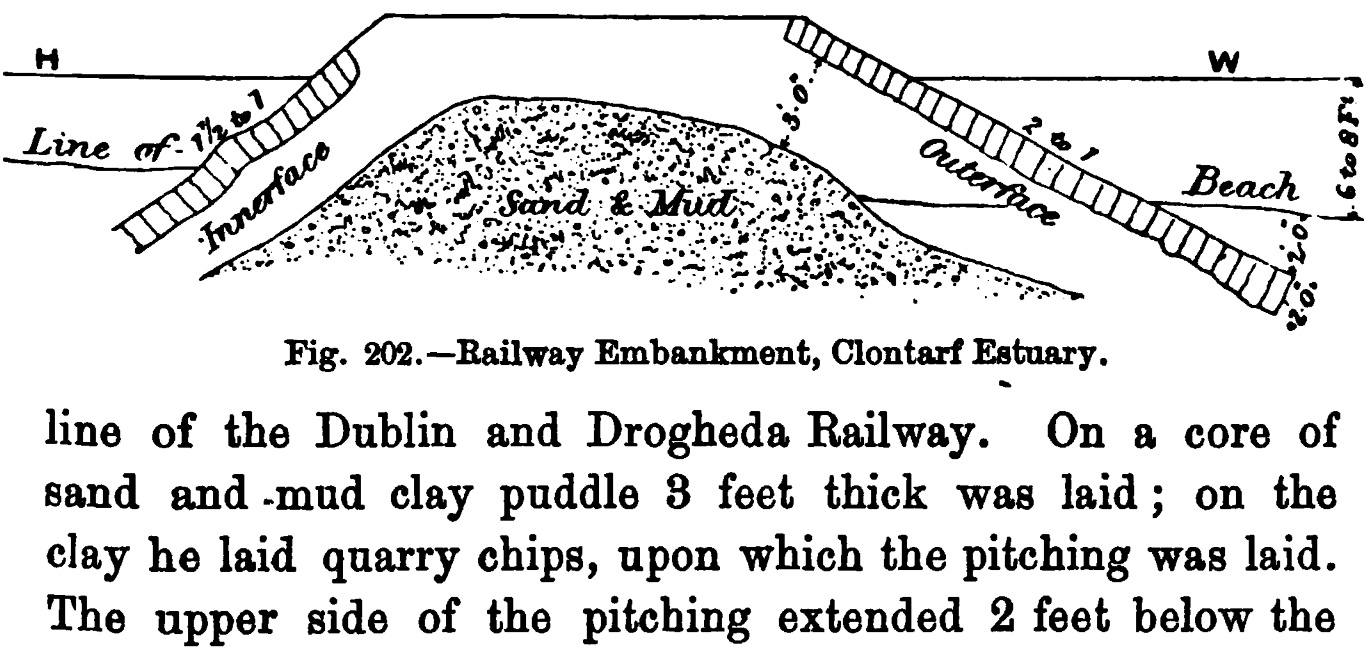


Fig. 202.—Railway Embankment, Clontarf Estuary.

line of the Dublin and Drogheda Railway. On a core of sand and mud clay puddle 3 feet thick was laid; on the clay he laid quarry chips, upon which the pitching was laid. The upper side of the pitching extended 2 feet below the surface. The pitching consisted of stones not less than 2 feet deep at the base, and 18 inches upwards. The slopes have stood well; though the middle has subsided considerably.

For another embankment on the same railway, crossing the Malahide Estuary, nearly one mile in width, the section

* See Mr. Brunlee's paper on Sea Embankments, in the *Proceedings of the Institution of Civil Engineers*, vol. xiv. p. 239.

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shown in Fig. 203 was adopted, in which the sea-face is of an ogee form, having reverse curves. This form was certainly not so good as that of a straight slope. The action of the waves tended to beat down the stones, resting only on their edges, on the most nearly level portion between A and B, and to work up the clay through the joints. The stones began to sink, the concave portion became dangerously hollowed, and the parapet

Fig. 203.—Railway Embankment, Malahide Estuary.

Fig. 204.—Dymchurch Sea Wall.

wall necessarily suffered. Groins are placed at intervals on the sea side of the bank; they have proved of utility.

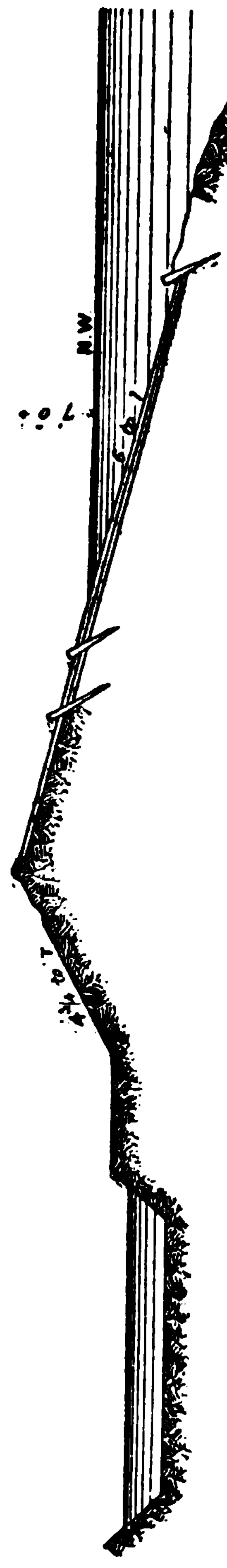


Fig. 205.—Sea Wall, St. Mary's, Kent.

Dymchurch wall, dyke, or sea embankment, on the coast of Kent, 3¾ miles long, was erected for the defence of Romney Marsh. It is shown in section, in Fig. 204, as remodelled by Mr. Walker. It has an uniform slope of 6 or 7 to 1, with a curved summit, to a radius of 7 feet, in order to throw off the water from the top of the wall. Paving-stones of Kentish rag are bedded on clay. Previously, the slopes were too steep, and the clay was washed away by the receding waves penetrating through the joints. By the flatter slope, the power of the ascending waves is gradually spent. Plank sheet-piling was driven along the toe of the bank, and rows of the same were driven down the face of the work transversely. By the transverse sheet-piling, breaches, where they occur, are limited in extent. The crown of the dyke is 20 feet wide, and the land slope is 4½ to 1.

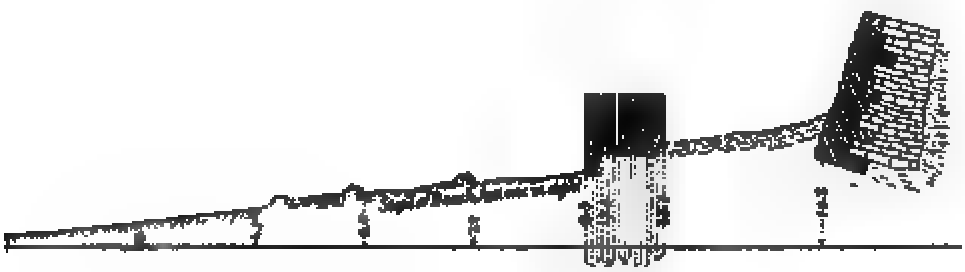
In the estuary of the Thames, the sea-banks are formed like the Dymchurch wall, but lighter and steeper, as exemplified at St. Mary's, Kent, Fig. 205, where the sea slope is 5 to 1, and the land slope 4¾ to 1.*

Vertical walls, constructed as sea-defences in shallow water, are, as befor remarked, more troublesome to keep in

* See "Chatham Lectures—Marine Engineering," by Mr. J. B. Redman. 1877.

good order than sloped work. Mr. R. Stephenson constructed sea-walls on the line of the Chester and Holyhead Railway, near Conway. These walls, 1¼ mile in length, extend over portions of the line which form the terrace beneath the steep slope of Penmaenmawr. The average section of the main sea-wall, as actually constructed, is shown in Fig. 206; 5¼ feet thick at the top, and 14 feet above high-water spring-tides. The parapet was carried 8 feet high above the level of the rails, with a curved overhanging face, to a radius of 25 feet, so as to throw off the waves. The wall was, for half the thickness, of dressed ashlar, the remainder of coursed rubble. As the beach was found seriously to subside at the base of the wall, a low terrace of

Fig. 206. - Sea-wall, near Conway.



Figs. 207.—Breakwater for Sea-wall.

large stones and a breakwater, Figs. 207 were formed in front of the wall, to preserve the foundations, and, at the same

time, to break the waves and prevent the projection of solid water over the parapet. The breakwater consisted of a zig-zag row of piles, 2 feet clear apart, in right-angled bays, four piles to each bay, the inner angles being from 12 to 15 feet from the face of the wall. The piles were backed with 3-inch planks.

But though slopes are better than vertical, or nearly vertical faces, in shallow water, the vertical face is preferable in deep water. The preference is clearly established in the practice of construction of piers and breakwaters.]

CHAPTER VI.

HARBOURS.

THE construction of piers, jetties, and breakwaters is much influenced by the considerations of the effects of winds, tides, and currents, above described. The objects they are proposed to effect are to procure tranquillity in the interior of ports, and at the same time to facilitate the manœuvres of shipping entering or leaving them. To explain the details connected with such works, it becomes, therefore, necessary to dwell somewhat at length upon the general character and description of ports to which they form such important adjuncts.

It is usual to designate by the term "port" a space of water in connection with the sea, of variable dimensions, and of depths either constant or variable, in which ships may obtain shelter from tempests or from an enemy, repair any damage they may have received, or discharge and replace their cargoes. Such ports may be either upon the immediate sea-coast, or at some distance from the mouth of a river. The former, again, may either possess a broad expanse of sheltered sea, or, as they are called, roads; or they may be placed upon unprotected open shores. Roads also may be natural or artificial, open or sheltered, according to the configuration of the coast and of the bed of the sea in the particular direction under consideration. But of whatever description they may be, their utility mainly depends upon their possessing a sufficient depth of water for a vessel to ride at anchor at any time of the tides; whilst the bottom must be suffi-

ciently firm to allow the anchors to hold in a storm. If such roads have convenient access, they facilitate commerce by forming calling stations where vessels may wait for orders, or where they may wait for the winds or tides requisite to carry them into harbour.

Both roads and ports may be further sub-classified into those without tides and those with tides, according as they may be situated upon waters affected or not by that semi-diurnal inequality. The ports upon the Mediterranean, the Gulf of Mexico, the Caspian, and the great fresh-water lakes, may be considered as of the former class; those upon the shores of the ocean as of the latter. Practically, also, the usages to which roads and ports are specially appropriated give rise to a further separation into the respective classes of commercial and military.

The variable degree in which harbours are affected by the tides produces a marked difference in the influence of roads upon their utility. For if the interior of the harbours should not possess a sufficient depth of water at low tides to allow vessels to enter, evidently it will be necessary that these should wait until a favourable moment should arrive, and this could only be effected in a road close to the mouth of the harbour. As the larger class of vessels are not constructed to take the ground, as it is called, or to lie high and dry between the intervals of the tides, it becomes necessary to construct floating docks to receive them in all ports so circumstanced.

The utility of roads, whether open or sheltered, must of course depend upon the number of vessels they are able to accommodate. Sufficient space must be left round each vessel to allow of its swinging upon its anchor, according to the changes of the winds or the tides, without its being exposed to foul any other vessel. It is usual to calculate upon a radius of two cables' length for ships of war, and of about half that radius for merchant vessels.

On account of the great space thus required, roads cannot be entirely formed by artificial means; they must exist naturally, in a more or less perfect state. It is, however, possible to improve such as may exist by the construction of breakwaters or of jetties, so as to shelter any portion exposed to the violent action of storms; or by dredging any shoals to increase the available surface for anchorage. The roads of Plymouth, Cherbourg, Cette, and at the mouth of the Delaware in the United States, may be cited as illustrations of what has been done to create an artificial shelter; whilst the port of Nieuwe Diep, on the Zuiderzee, and of Lorient, may illustrate the methods employed to clear an already existing roadstead. There were, however, some peculiarities attached to the amelioration of the port of Nieuwe Diep which require to be noticed hereafter a little in detail.

The most advantageous situation for a port is at the extremity of a roadstead, especially if the channel of communication assumes a tortuous form. Should this not be the case, it will be necessary to construct piers, jetties, or moles, to destroy the undulations communicated by the open sea. It is also of vital importance for the security of a port that it should be surrounded by high lands on the in-shore, to guarantee vessels from the effects of the winds blowing from that direction.

As the piers at the mouths of harbours are constructed with a view to facilitate the manœuvres of vessels, it is found preferable to make greater provisions to assist their entry than their departure. A vessel in harbour can always wait for favourable weather; whilst those coming in from the open sea may often be driven in by stress of weather. In all tidal harbours, also, it is necessary to place the piers so that they may coincide with the direction of the currents, and in such a manner that the ships should not be carried against them whilst passing to the interior. Of course this pre-

caution will require to be observed in tideless harbours, if any decidedly marked littoral current should exist; but it is of more serious importance where the currents flow in alternate directions than where they are permanent. It is to be observed, however, that the manner in which vessels are towed out from the interior of a port will influence the form of the piers to a certain extent. Because if the vessels are towed in by men, or warped in by a rope, the piers must be carried out so far that they should be able to make their first tack without falling on the opposite side; whilst if the vessels be towed out by steamers, the extension of the piers need only be regulated by the necessity for protecting the entry of the harbour from the effects of the currents.

Beyond the jetties, in all seaports of importance, it is usual to construct an outer harbour, surrounded by quays, at the bottom of which are placed the docks intended to receive vessels of large tonnage, and to retain the water entering at high tide, so as to allow the operation of unloading to be performed whilst the vessels are afloat. Small vessels and fishing craft generally remain in the outer harbour, as do also the coasters or other craft able to depart at half-tide. The sluice-gates, and the leading channels from the back-water, whether it be natural or artificial, are also placed upon some portion of the outer harbour.

The length to be given to this part of a port will depend very much upon the facilities it may possess, with respect to the entrance or departure of vessels. It frequently happens that during windy weather these may retain a very considerable portion of the momentum derived from the velocity they had whilst under sail; sufficient space must therefore be allowed for the destruction of this momentum before the vessels arrive at the locks giving access to the inner harbour. On this account, also, it is preferable to direct the entrance between the jetties a little out of the line of the prevailing winds, and to place the lock-gates somewhat out of the direct

path followed by the vessels entering. Generally speaking, it will be found sufficient to make the length of the outer harbour about 700 yards.

The width usually given to the channel between the jetties at the entrance of a harbour is that which shall be sufficient to allow the passage of three of the largest ships frequenting it, under sail, and at the same time. The minimum may vary between 100 and 240 feet, according to the size of the ships and to the power of the sluices. At the extremities the width should be increased, because the ships require more room to perform their evolutions whilst under the influence of the way they carry in from the open sea than when they have followed the narrow channel for some time. The introduction of steam-tugs has called for a greater width of channel than was required under the old system.

The dimensions and dispositions of the inner harbour will be regulated by the nature of the vessels frequenting it, and the depth must be such as at all times to exceed the draught of water of the largest vessel it is likely to receive. This excess of depth is required, firstly, to compensate for any loss of water which may take place between two consecutive tides, whether it arise from evaporation or leakage; secondly, to allow of the abstraction of a certain quantity of water for the purpose of scouring the passage in front of the lock-gates; thirdly, to prevent the possibility of the vessels grounding, should any agitation either be transmitted from the outer harbour, or be produced in any manner in the inner one; fourthly, to allow of the gates being opened a little before the period of the highest tide so as to permit vessels of light draught to leave at an early period; and fifthly, to provide against the gradual silting up of the inner harbour. In order to provide against all these sources of possible annoyance, and to secure the eventual advantages, it is usual to make the depth of inner harbours 2 to 3 feet in excess of what is absolutely required to receive the vessels frequenting it.

As an economical question, we may consider that the dimensions of length and breadth should be decided upon the principle that the greatest number of vessels should be able to lie alongside the quays, with the smallest amount of expense in the excavation of the water surface. At the same time it is necessary to leave between the different tiers of vessels a space sufficient for the evolutions of those about to enter or depart. In some of the most convenient modern ports, intended to receive commercial vessels only, the length is made about five times the width, and in these instances vessels often lie in three tiers on either side. In military ports, however, it is necessary to bring every vessel close to the quay, and therefore the proportionate length may be less than that above stated. At Cherbourg, where the expense of excavating the basins in the solid rock was enormous, their dimensions were reduced to the minimum; one of them was therefore made nearly square, or 754 feet wide by 852 feet long; whilst the second was made 656 feet wide by 1,312 feet long, or in the proportion of 1 in width to 2 in length.

The gates closing the inner harbour should be constructed with a view to secure the most rapid operations in opening and shutting, and to render the leakage at the several joints as small as possible. Their width must be regulated by the class of vessels entering. Barques and sailing vessels under 500 tons burthen do not require a greater width than 50 feet; a first-class frigate does not require more than 52 feet; nor does a first-class sail of the line require more than 60 feet. But the colossal dimensions of some of the modern steamers render it necessary to make the gates through which they are to pass not less than from 70 to 80 feet wide. It is customary to place sluices at the bottom of the lock-gates to assist in scouring the platform at the entry.

The depth of the floor of the passage will be regulated by the draught of water of the vessels entering. A 500-ton vessel will draw about 16 feet, a first-class frigate about

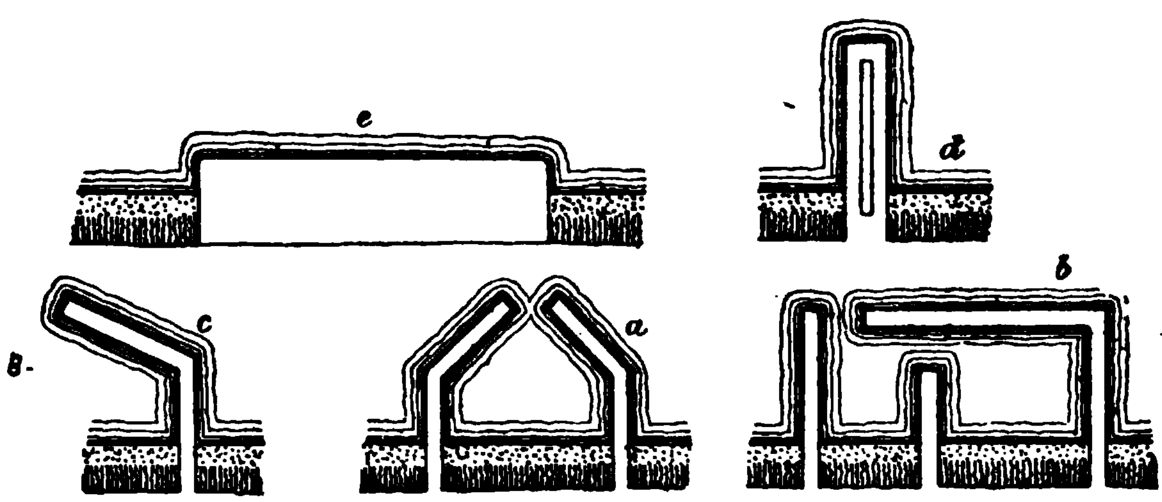
22 feet, and a first-class man-of-war about 27 feet, supposing them to be under full load. As the draught of steamboats is in nowise commensurate with their width, they may be considered to be comprised within these dimensions.

Unless there be very great inequalities of the tide, one set of gates will be sufficient to retain the water during the period of the ebb; but it frequently happens that it is indispensable to place a set of gates to exclude the flood-tide, especially when repairs are likely to be required to the flooring of the gate passage or the inner harbour. It appears, therefore, a very necessary precaution to construct the chamber and passage so as to admit of placing two pairs of gates, respectively to provide against the ebb and the flood tides; and in many commercial ports it may even be advisable to place a second pair of ebb gates in order to allow the intermediate space to become a species of lock to facilitate the departure of small craft at the half tides. A provision should also be made for the insertion of a coffer-dam; and in almost all cases it will be necessary to construct a turning bridge for the purpose of connecting the roads or quays on either side of the passage. In the side walls of the lock chambers it is also customary to construct culverts, provided with gates and raising machinery, for the purpose of assisting the scouring action of the sluices in the gates themselves upon the passage leading up to the latter. It follows, therefore, from the necessity for these several works, that the length to be given to the entrance from the outer to the inner harbours of any port must vary according to the local conditions of tide, or of the communications between the two banks of the inner harbour.

[Harbours admit of classification, as havens for the protection of ships during storms, or "harbours of refuge;" and as ports adapted for commercial purposes.

Harbours of refuge consist of one or more breakwaters, so placed as to form a safe anchorage-ground, easily accessible to the largest vessels in all states of the weather and at all times of the tide. A breakwater acts as a barrier to the progress of waves, and thus secures comparatively quiet water landward of it. Breakwaters are not used for commercial traffic, and they present nothing more than a mass of sufficient weight and solidity to break the force of the waves. The old breakwaters offer many instances of this kind. But the more modern breakwaters are commonly constructed with a parapet wall, which, besides of course contributing to prevent the waves from breaking over the top, also operate usefully as a protection from the wind.

Harbours constructed for commercial purposes are necessarily provided with breakwaters, piers, quays, or docks, singly or in combination, by which a sheet of water is enclosed and tranquillised, in order that vessels may be moored at the quay-walls or wharves forming the inner sides of piers or docks. Mr. Thomas Stevenson* illustrates the varieties of commercial harbours, as in Figs. 208.



Figs. 208.—Commercial Harbours.

Where the coast-line lies open to a heavy sea, it is often necessary to make a double harbour, *a* or *b*, where the entrance to the river basin is situated within the

* "The Design and Construction of Harbours," 1874, p. 4.

sheltered area formed by the outer works. The kanted or curved pier *c*, affords shelter to vessels lying under the lee of the kant, the sheltered side of the pier being finished as a quay. Piers may be formed with a double kant, like the letter T. A straight pier, *d*, generally projects at right angles to the coast-line, and, unless when the wind blows right on shore, the straight pier always affords shelter on the lee side. Both sides may be finished as quay walls, and the parapet, if there be one, is built in the middle of the roadway. The simple quay, or wharf, *e*, is constructed parallel to the line of the shore. It affords no shelter; it simply affords facility for vessels loading and unloading in deep water.

The width of the entrance of harbours varies between the limits of 100 feet and 1000 feet. But the usual variations range between 200 feet and 400 feet. The following area few instances given by Mr. Stevenson.

	Width of entrance.
Dover	120 feet.
Newhaven	150 „
Ramsgate	190 „
Portland	400 „
Ayr	215 „
Leith	240 „
Yarmouth	250 „
Sunderland	315 & 368 ft.
Aberdeen	378 feet.
Portsmouth	700 „
Kingstown	960 „
Calais	328 „
Boulogne	230 „]

CHAPTER VII.

BREAKWATERS.

The formation of a breakwater is sometimes necessary to secure the tranquillity of the roads. Plymouth, Cherbourg, Cette, and the port at the mouth of the Delaware, have been cited as illustrations of this species of construction, and they offer sufficient differences of principle even to require a somewhat detailed examination. A notice of the breakwater of the port of Buffalo, on Lake Erie, is added, for the purpose of showing the methods adopted by the American engineers in what may really be called their fresh-water seas.

The "Digue" of Cherbourg, the first in chronological order and in size, is unquestionably one of the boldest and most gigantic works executed by man. Its total length is about 4,120 yards, and it consists of two arms, respectively 2,441 and 1,679 yards long, forming at their junction an obtuse salient angle towards the open sea of about 169°, with an average depth of water, at high spring tides, of about 62 feet. The foundations for a circular battery, 100 feet in diameter, have been prepared at the east end, and at the west end for a similar fort of 134 feet in diameter; whilst at the point of junction of the two arms, the foundations have been laid for a fort of about 640 feet in development. The width of the passes between the extremities of the digue and the main land at the points where fortifications (crossing their fires with those of the intended forts on the digue) are erected, is respectively 1,040 and 2,515 yards. The area

sheltered by this work is about equal to 1,927 English acres at low tides; but of this, not more than 696 acres have a depth of 27 feet at the lowest tides. Of this deep-water surface it also appears that nearly two-thirds are exposed to the unbroken violence of the ocean during the winter months, so that really the roads of Cherbourg, notwithstanding the immense cost of the digue, can hardly be said to be able to shelter more than from 25 to 30 sail of the line, inasmuch as each vessel requires from about 8½ to 10 acres superficial to swing freely upon its anchors. In the shallower parts of the roads an equal number of frigates could be made to ride in safety.

The original intention in constructing the digue was, that it should be submersible at one-third of the rising tide. This intention was subsequently abandoned, and the height was proposed to be at different periods, firstly, that of the ordinary spring tides, then 10 feet above that line, and finally the actual height of 12 feet 6 inches was adopted. The sketch, Fig. 209, opposite will represent the normal section of the digue; the sketches, Figs. 210, 211, 212, are introduced to show the modifications of the profile superinduced by the action of the sea between the years 1788 and 1829. As will be seen upon inspection of these figures, the slope of the seaward face had materially changed; and in 1829, as it was found that the tranquillity of the roads was by no means secured, and that the small blocks were constantly swept over from the sea side to the inner face, it was resolved, after long and anxious deliberation, to crown the top of the sea slope with a vertical wall, as shown. The original digue was completed to the line of the low spring tides in small blocks, and after the materials thus added had been allowed to settle, they were covered with a bed of hydraulic concrete 5 feet thick; and upon this a solid wall of coursed ashlar masonry, the external and internal faces of which were executed in granite, with rubble hearting, was erected as

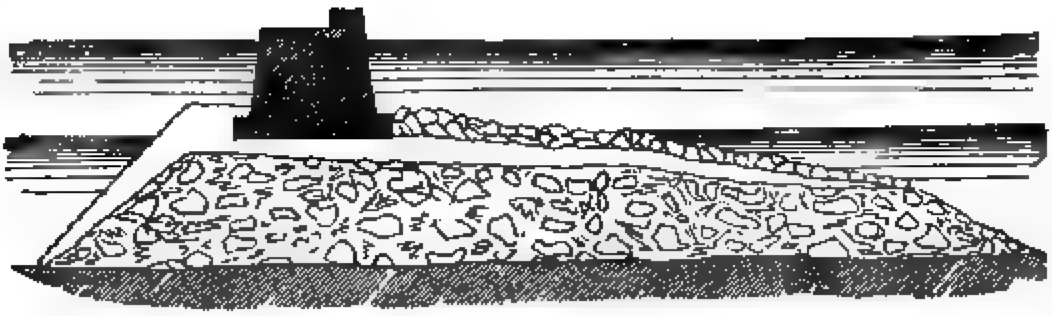


Fig. 209.

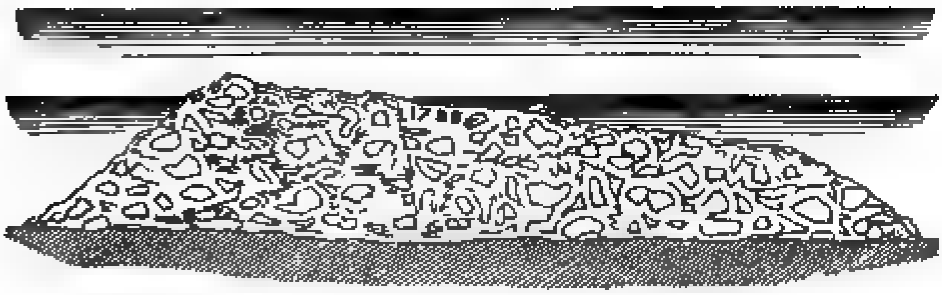


Fig. 210.

Fig. 211.

Fig. 212.

Digue at Cherbourg.

shown. The top of the sea slope is covered with large loose blocks, and at the extremities of the wings it is further protected by immense artificial blocks of about 40 tons weight each, formed of rubble set in Roman or Portland cement.

The breakwater of Plymouth is formed in a bay sheltered on three sides by land rising to a considerable height, and only open to the south. Several banks, or natural reefs of rocks, exist, between which and the shore there were three principal passes towards the east, the west, and in the centre. The breakwater is erected upon the banks situated the nearest to the interior of the harbour, and closes the centre passage; the banks situated more towards the open sea serve to break the fury of the waves before they arrive upon the breakwater.

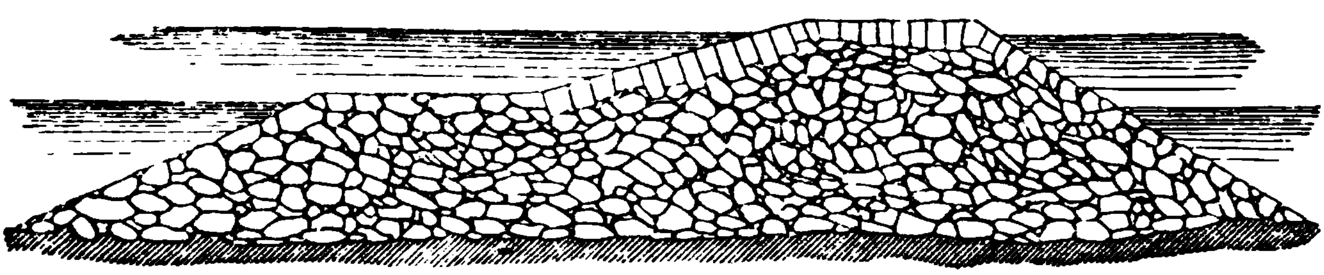


Fig. 213.—Plymouth Breakwater.

The main body of the breakwater is placed perpendicularly to the S.S.E., from which quarter the severest storms assail the Plymouth Roads. The total length is 1,700 yards, of which the rectilineal central part occupies 1,000 yards, and the two arms, forming on either side angles of about 135° with the centre, occupy respectively 350 yards each. A surface of about 1,120 acres is, by means of this work, rendered available for large vessels.

Originally it was intended to make the width of the top of the breakwater only 11 yards, and that of the bottom about 55 yards; but during the execution of the works the width of the top has been increased to 15 yards, and that of the bottom to 133 yards. At the level of the low water at spring tides a set-off 22 yards in width is formed, and the slopes

from this point upwards, on the sea side, are paved with large stones, 4 feet by 3 feet 6 inches, by about 3 feet thick, laid with an inclination of 5 base to 1 height and bedded in Roman cement; and it is proposed to continue this paving below the lowest water line by means of the diving-bell. The height of the crown of the breakwater is only 2 feet above the level of high spring tides.

It appears to be beyond question that the long slope of the Plymouth Breakwater is less exposed to be injured by the violent shocks of the sea than the vertical wall of the Cherbourg Digue; but at the same time it is equally beyond question that the latter destroys far more effectually the agitation and undulation of the open sea, and offers a greater resistance to their transmission into the inner harbour, because the waves in Plymouth Sound during violent storms break over the slopes, whilst at Cherbourg all their effect is destroyed by the wall. In the latter case, however, the descending motion of the return wave is materially interfered with. The vertical wall at the top of the long slope transforms it, in fact, into a horizontal motion, whose velocity is highly dangerous to the stability of the foundation of the wall. It is also found that the large blocks of stone detached from the outer slopes are driven against the outer face of the wall with extraordinary violence during great storms, whilst upon the long paved face of the Plymouth Breakwater the waves, not meeting with any abrupt resistance, break in precisely the same manner upon the incline that they would do upon a natural shore, and with a considerably diminished degree of violence. It is true that, in consequence of this form, they acquire an increased horizontal velocity in their original direction; but as the top of the slope is rendered as smooth as possible, there are no salient points in the masonry able to attract, as it were, the destructive action of the waves. Notwithstanding the precautions observed in the execution of the top slope of the breakwater, it is by no

means of rare occurrence that blocks weighing from 2 to 5 tons are carried over from the sea to the land side; and in February, 1848, considerable damage was caused to the upper parts.

The breakwater in Delaware Bay was designed not only to form an artificial roadstead sheltered from the effects of the prevailing winds, but also from the drift ice brought down occasionally in large quantities from the upper parts of the Schúylkill and Delaware Rivers. It was also contemplated that the port thus created would rather be a place of refuge for ships bound coastwise, than it would become a touching port for vessels dropping down the river. In consequence of these local circumstances, the works for the shelter of the roads consist of a breakwater and what may be called an ice-break. The breakwater itself is in a straight line, in a direction W.N.W. to E.S.E., and of a total length of 1,000 yards measured upon the line of high water, leaving a channel of about 1,000 yards in width between its E.S.E. extremity and the main land. At a distance of 555 yards from the W.N.W. extremity, the prolongation of the line of the inner slope of the breakwater meets the line of the inner slope of the ice-break, forming with it an angle, towards the shore, of 146° 15′. From the point of intersection, the line of the ice-break is carried respectively 272 yards W. by S., and 228 yards E. by N., making a total length of about 500 yards, with a clear passage of 350 yards between it and the main breakwater.

The space thus sheltered has an area of about $\frac{1}{8}$th of a mile, so far as the waves raised by winds from the N.W. to the E., passing by the N., are concerned; and a space of $\frac{7}{25}$ths of a square mile so far as those caused by winds from the N.W. to the E. (by the N.) are concerned; and in both cases the minimum depth so sheltered is 24 feet. The area of sheltered road, with a minimum depth of 18 feet, is about $\frac{7}{10}$ths of a square mile. The tides in this locality are but feeble, for the

average range of the neap tides is about 4 feet 8 inches, and that of the equinoctial spring tides about 7 feet 6 inches; whilst the greatest range that has been noticed has never exceeded 8 feet 10 inches vertical.

The transverse section of the breakwater was made as follows:—The inner slope, towards the harbour, was formed at an angle of 45° with the horizon; the top was made 30 feet wide, and at 5 feet 4 inches above the level of the highest spring tides. The outer slope was carried down, with an inclination of 3 base to 1 in height, to a depth of about 19 feet from the highest spring tides, and from thence to the bottom, at an angle of 45°. The mass of the work between the sea bottom and a horizontal plane passing at 6 feet below the lowest spring tides was formed of stones weighing from

Fig. 214.—Breakwater, Delaware Bay.

¼ to 2 tons, and the slopes covered with blocks of from 2 to 3 tons minimum weight. Between this point and the plane corresponding with the lowest spring tides, the body of the work was executed in stones weighing from ½ to 2¼ tons, protected externally by blocks weighing 3 tons each at least; and the upper portion was formed exclusively of blocks weighing from 4 to 5 tons, laid as regularly as possible, the slopes being covered with the largest blocks, laid as headers.

The breakwater of Cette, Fig. 215, is principally remarkable on account of the great height to which it is carried above the highest water line; this is not less than 19 feet. The total length is about 514 yards, and the outline on plan is convex towards the open sea. During its execution, observations were made from which it may be inferred that the constant

undermining of the sand upon which this breakwater was constucted, so long, at least, as the transverse profile was made very steep towards the open sea, would indicate a danger of superinducing a ground-swell highly injurious to the permanent solidity of the works, unless the sea slope in similar cases were carried out at once to the full width. It is also seriously questioned by the pilots resorting to this harbour, whether the breakwater does not materially assist the natural tendency to silt up which so strongly marks this and several other ports of the Mediterranean.

Fig. 215.—Breakwater, Cette.

The breakwater upon Lake Erie, Fig. 216, at the entrance of the port of Buffalo, in the State of New York, is constructed with nearly as much solidity as similar works upon the shores of the ocean. Its length is 484 yards in a straight line; the platform at the level of the first set-off is 18 feet wide, and 5 feet above the water-line in the interior. A wall 5 feet high is carried up above this platform, and beyond this a gentle slope of about 8 base to 1 in height is carried down to the bottom of the lake. Towards the port the face of the breakwater is perpendicular, and it is defended from being injured by vessels lying alongside it by guard-piles driven in

every 5 feet apart. A row of sheeting piles is driven upon the external edge to protect it from the effects of the ground-swell; the mass of the breakwater is executed in loose rubble masonry.

In addition to what has already been said with respect to the precautions requisite to be observed in the construction of piers and breakwaters, it is important that the lower courses of the masonry be covered by the succeeding courses as rapidly as possible, not only to enable them to resist the direct action of the waves, but also the syphonic action of the

Fig. 216.—Breakwater, Lake Erie.

water driven into the joints. For the latter reason also it is important that the joints between the stones be executed with the most energetic cements, and be made to fit very closely. If the hearting be executed with small stones, the inequality between its rate and degree of compression is likely to give rise to hollow chambers, which facilitate this syphonic action; and it appears therefore indispensable to introduce a greater or less number of horizontal bond courses, according to the nature of the materials employed.

Experience also appears to show that it is safer to raise the masonry, in such positions, to its full height partially at once, rather than to endeavour to carry it up regularly thoughout the whole length, in order that the superincumbent weight of the upper courses may assist in maintaining the lower ones in their places. This precaution is peculiarly necessary when the walls have reached the mean level of the sea, at which point the waves act with the greatest effect. In solid masonry in

these positions, the rapid setting of the cements or mortars is a condition of vital importance; and it is also necessary that only such materials of either of these classes be employed, as allow of being prepared for use with salt or fresh water indifferently.

[Breakwaters, like sea defences, are to be constructed with reference to the conditions of the site. In profile and construction, breakwaters are of three types:—A mass of small rubble with varying slopes, and paved at the top, as Plymouth breakwater; a rubble mound, with a built-up pier founded on the summit; a wall upright, or nearly so, brought up from the bed of the sea. Engineers have been too much in the habit of ignoring principles in the designing of breakwaters, yielding an unqualified obedience to "circumstances," according to what Mr. Hawkesley calls the "rule of thumb and scowl of brow" system. It is better not to affect self-abasement on points of principle, for a lack of command of principle signifies a lack of command of practice. Mr. Abernethy justly remarks that, although local circumstances may vary, the hydraulic laws which regulate the motion of waves are fixed and immutable, and that a definite conclusion can be arrived at as to the best form of breakwater for the deep-water oscillating wave and for the shoal-water wave of translation or percussion. The variety of opinion applies mostly to the degree and extent of slopes, and to the relative merits of slopes and vertical walls. Long rubble slopes, say of 7 to 1, between the levels of high water and low water, which convert the deep water oscillating wave into a wave of translation, is an error of construction, not only with respect to original cost, but also as to the cost for maintenance in the future. The long seaward slope is exposed, not only to the percussive action of the waves of translation, but also to the recoil of the sea, or back draft,—what sailors call the "undertow" of the wave. The effect

of such constant action must be the conversion by attrition of the face on the seaward side into a mass of boulder stones. After a storm, the stones would be disturbed and drawn out, and the slope lengthened. To preserve this portion of the work, it would be requisite to pave it with massive ashlar pitching; but then, again, the waves would by such facilities for translation retain still greater force for advancing upon the vertical superstructure. These are simple ruling principles.

Mr. John Murray discusses and compares the designs and costs of breakwaters constructed on different systems. It is generally admitted, with respect to work under low water, that rubble-stone, used as *pierre-perdu* (literally "stone at random"), piled up from the bottom of the sea, remains stationary until it arrives at a level of 12 feet or 15 feet under water; and that the slopes of the deposited mass of broken stone assume an angle of 45°, or an inclination of 1 to 1. This is the least expensive method of bringing up the works around the coast to that level, as is readily proved by comparative estimates. Mr. Murray takes, for example, a simple form for the transverse section of a mole, brought up from the bottom in a depth of 6 fathoms, with a spring-tide rise of 16 feet, according to three systems of construction, Figs. 217, 218, and 219. The coping is taken at 10 feet above high water, without any parapet. The thickness of the outer wall at the top is 7 feet, with a batter of 3 inches to a foot; and that of the inner wall is 6 feet, with a batter of 1½ inches. The mole is 40 feet wide at the top.

To bring up the work to the height of 12 feet below low water, as shown by sectioning in Fig. 217, in freestone set in mortar, by means of the diving-bell, &c., there would be consumed 2,286 cubic feet of freestone work, at, say 2s. 6d., costing £285 15s., and 57⅘ cubic yards of rubble, one-fifth being deducted for interstices, at 3s. 6d., costing £10 1s. 5d.; making together £295 16s. 5d. In the second case, where

the substructure consists of a simple deposit of rubble-stone, allowing a berm 10 feet wide at each side of the superstructure, as in Fig. 218, there would be required 209$\frac{4}{5}$ cubic

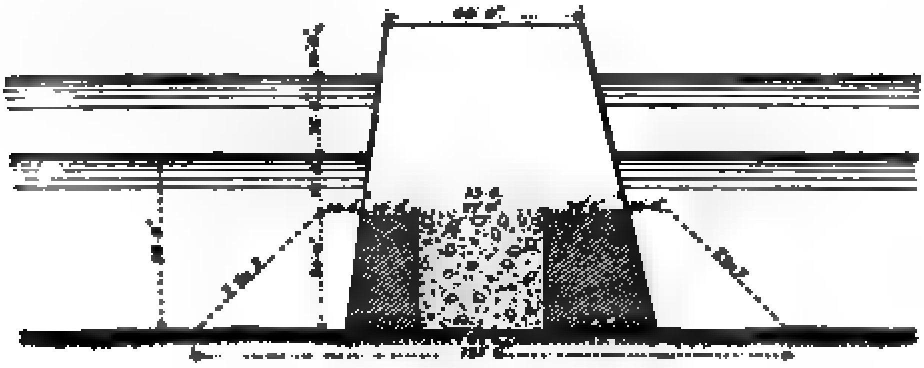


Fig. 217.

Fig. 218.

Fig. 218a.
Types of Breakwaters, by Mr. J. Murray.

yards of rubble filling, deducting one-fifth for interstices, at 3s. 6d., costing £36 13s. 5d. per lineal yard. These estimates are exclusive of staging, and they are as 8 to 1. The

cost of the upright mole would be considerably augmented if the facing of the walls be of granite, and the interior filled with level courses of concrete blocks, the system of construction adopted for the pier at Dover. There is, therefore, good ground for preferring the simple rubble deposit.

Adding the cost of the superstructure of the simple form shown in Fig. 218, with connecting walls from low water upwards, 6 feet thick, and at central intervals of 20 yards—estimated as £262 6s. per lineal yard—the total cost of the mole would be—

	£.	s.	d.	
Upright from the bottom	558	2	5	per lineal yard.
On a rubble stone deposit as described	298	19	5	„

But economy may be still further promoted in the construction of breakwaters, designed as such, and nothing more: dispensing, wherever it is possible, with skilled labour, and defending the sea-face solely by masses deposited to find their own slope, and of sufficient gravity to withstand the action of the waves. As the action of the waves is a maximum at the surface, and diminishes as the depth increases, the smaller stones should be deposited at the bottom, and the largest stones, or blocks of concrete, towards the top. Moreover, the large and the small stones should not be mixed, for small pieces of stone mixed with large rubble, far from consolidating the work, very often have the effect of facilitating the displacement of the larger masses. Stones weighing 5 tons, or even 7 tons, have been thrown out of place in consequence of small stones getting under them and between them, and keeping them in motion during storms. Taking a minimum depth of 22 feet under low water as the limit of the injurious action of waves, the nucleus, or core of the breakwater, from the bottom up to that level, A, Fig. 218*a*, may be wholly composed of third-class rubble, comprising stones of from quarry rubbish up to

blocks of half a ton in weight. On this may be laid a coat of second-class rubble, B, in blocks of from half a ton to 2 tons in weight, which, it is considered, would remain stationary up to a level 12 feet below low water. Then a stratum of first-class rubble, C, from 2 tons to 5 tons in weight, having a bench or berm on the sea side; to receive the upper part of the breakwater, D, consisting of blocks of béton or concrete of from 20 to 25 tons in weight, terminating at high water, capped with two or three courses of heavy blocks, to a height of 10 or 12 feet above high water, bonded and adjusted by manual labour. The slopes of the section Fig. 219, proposed by Mr. Murray, are 1 to 1, except for the seaward side, above low water, which is 2 to 1. The quantities and cost are estimated as follows:—

$145\frac{2}{9}$ cubic yards third-class rubble, A (deducting $\frac{1}{6}$ for interstices).					
$121\frac{7}{9}$ „ „ second „ „ B „ $\frac{1}{5}$ „					
$98\frac{2}{9}$ „ „ first „ „ C „ $\frac{1}{4}$ „					
$365\frac{2}{9}$ . . . . at 4s. 6d.	.	£82	3s.	6d.	per lineal yard.
2,568 cu. ft. concrete blocks, D, deducting $\frac{1}{3}$ for interstices, at 8d.	.	80	12	0	„
760 do. do. E, at 9d.	.	28	2	6	„
		190	18	0	
Use and waste of staging	. .	10	0	0	
		200	18	0	

This sum contrasts favourably with the estimate costs of the other structures, Figs. 217 and 218.

It may be remarked that there is a considerable saving of material in the use of large blocks of concrete in heaps, by the intersticial vacancies which exist between them, amounting to one-third of the gross volume of the heap. The interstices, besides, are useful in providing a passage for the waves, and at the same time breaking their force.

French engineers have highly appreciated the advantages of the system of construction above noticed. The new mole Algiers was at first constructed wholly of large blocks of

concrete, about 13 cubic yards in volume, weighing about 25 tons each, in water averaging 50 feet deep. But, in prolonging the mole, since 1847, a less costly system was adopted, according to Fig. 219. Small rubble was deposited

Fig. 219.—Mole, Algiers.

on the bottom, up to a level 33 feet below the water line, to the natural slope of 1 to 1; so economising, to a great extent, the cost for the substructure. Upon this base the remainder of the work, consisting of blocks of concrete, was deposited to a slope of 1½ to 1 on the seaward side, and of 1 to 1 landwards, finishing at the water line at a width of 46 feet. The cost of the mole amounted to £366 per lineal yard.

The mole of La Joliette, Marseilles, in 35 feet of water, was constructed according to the separate system of rubble foundation—separating large from small stones. It is shown in section in Fig. 220; and Fig. 221 is a diagram showing the method of construction. The core A is a mass of small stones; B is a layer of larger stones of 11 cwt. to 1¼ tons; C is a layer of stones of from 1¼ to 3½ tons; and the layer D is of stones of from 3½ to 8 tons. The hearting, E^1 and E^2 is composed entirely of small stones. The outer face is covered with blocks of concrete, which, when deposited in the sea, assumed a slope of 1 to 1, at which they were not disturbed even during the heaviest gales. Above the water line the blocks were laid at a slope of 2½ to 1. The blocks were composed of hydraulic lime and sand as mortar, in the

proportion of one part to two parts of stone. The stone for the blocks was broken into fragments of 2 inches in diameter. They cost, in masses of 10 cubic metres, 20 francs per cubic metre deposited, or about 5½d. per cubic foot. The cost per block of 25½ tons weight was £8.

The quay of La Joliette is 60 feet in width, formed on the rubble deposit and paved on a bed of concrete. The inner wall of the quay was built in béton, sunk in caissons, which

Fig. 220.—Quay, La Joliette.

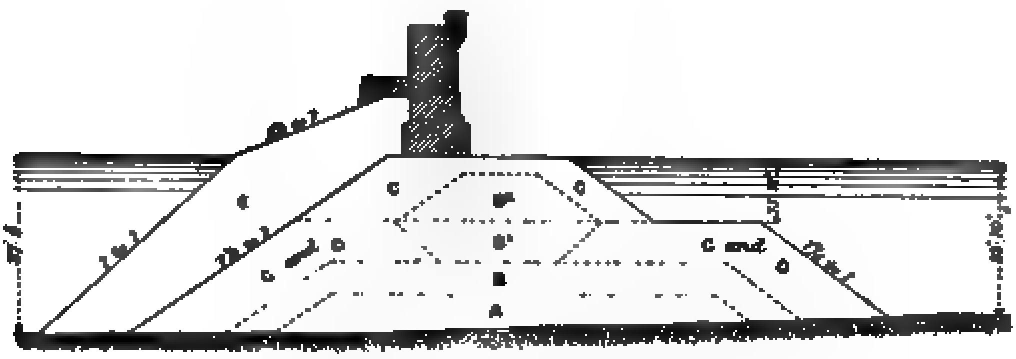


Fig. 221.—Quay, La Joliette.

rested on the berm of the rubble deposit. The cost for the rubble deposit, including the external coating of blocks of concrete, amounted to nearly £154 per lineal yard. Add for the parapet, with ashlar stone dressings, £61 10s., and the sum, say £215 per lineal yard, was the net cost of the structure as a breakwater simply. The additional cost of making the inner side effective as a commercial quay was £44; making the gross cost £259 per lineal yard.

Taken broadly, the quay of La Joliette has evidently been much less costly than the mole of Algiers. The difference is attributable to the prevalence of rubble work in La Joliette.

The only instance in the United Kingdom, so far as the writer is aware, in which large blocks of concrete have been employed for the defence of breakwaters, is that referred to by Mr. B. B. Stoney, who has deposited blocks of 140 tons in weight on the foreshore of a breakwater on the south side of Dublin Harbour. The foreshore had previously been constructed of large blocks of granite, weighing from 6 tons downwards, which had to a great extent been comminuted and gradually washed away. He replaced the stone blocks, in 1862, by blocks of concrete 50 tons in weight, which had lasted twelve years. But one of the blocks was, in the winter of 1873, moved over a distance of 30 feet, and turned upside down; and Mr. Stoney proceeded to lay blocks of the weight of 140 tons, to prevent the chance of displacement.

The essential value of large blocks as a covering for long-slope breakwaters, and of the separation of small rubble from large rubble, has been demonstrated, by contrast, in the experience of long-slope breakwaters protected by ordinary rubble. The breakwater, or pier, at Alderney, affords an instructive example in point. It was commenced in 1847, and completed in the end of 1864. The rise of spring tides is 17 feet, and the depth at low water increases from 45 feet near the shore to 133 feet at the head of the pier. The total length of the breakwater is 4,700 feet. On a long-slope base of *pierre-perdu*—hard stone from Mannez quarry—large and small, the superstructure for the most part consists of a sea-wall and a harbour wall, with filling in the intermediate space, surmounted by a promenade. It was constructed with a few modifications of structure as the work advanced. The last 2,000 feet were constructed to the

sections, Figs. 222 and 223, to the second of which the final length of 66 feet was constructed, constituting the head of the pier. It had been found that the rubble stone was not disturbed by waves at a greater depth than 12 feet below low water, and the foundations of the superstructure were carried to that level. The quay, Fig. 222, is 20 feet wide, and the promenade is 14 feet wide, making together a width of 34 feet. The faces were built with a batter of 4 inches in 1 foot. The quay stood 6 feet above high-water level,

Fig. 222.—Alderney Breakwater.

Fig. 223.—Alderney Breakwater.

and the promenade was 10 feet higher. The total width of the foundation was 59 feet. The lower courses of the walls were composed of Portland-cement concrete blocks, faced with granite on the seaward side. The hearting consisted of solid rubble in cement. It had been intended to make the rubble mound of such a form on each side of the superstructure as should remain undisturbed by the sea, rising at the face of the seaward wall to the level of low water, and on

the harbour side to 6 feet below low water. The natural slopes on the seaward side were assumed to be 5 to 1, from the wall to a depth of 15 feet below low water, changing at that depth to an inclination of $1\frac{1}{2}$ to 1 carried down to the bottom. On the landward side, the slopes were designed to be $1\frac{1}{4}$ to 1 to the bottom, except a short length of less inclination next the wall, to a depth of 9 feet below low water. The slope on the seaward side was altered by the action of the waves and flattened to 7 to 1, and in deep water the flatter slope extended to a depth of 20 feet below low water, where the sea ceased to act on the mass, and the change of slope takes place to a steeper inclination. It was found impossible, moreover, even with the large quantities of rubble deposited for the renewal of the foreshore, to maintain the level of the mound close to the sea-wall at the level of low water. The depression of the surface is exemplified in Fig. 222, where the dot-line indicates the original surface of the mound. The disturbance extends to a distance as much as 80 or 90 feet from the seaward wall, at the maximum depth, 20 feet. The great distance and depth of the disturbing action are attributable, no doubt, to the recoil of the waves from the wall, as each wave, during a storm, rises to a great height above the breakwater, then falls and rushes down the slope to the mound, opposing an overpowering resistance to everything in its course, till it rebounds some height into the air on meeting the next wave, at a distance of about 70 or 80 feet from the wall.

Breaches were made in the superstructure at various points—mostly in the deeper water. The rubble was dashed against the seaward wall, and, no doubt, helped to open the breaches. The superstructure subsided unequally at different portions, longitudinally as well as transversely, and cracks and openings were formed in the interior; so that the action of the water entering these openings, in conjunction with the confined air, under the pressure of the waves, aided in

producing cracks. That subsidence, in a considerable degree, should have taken place under the weight of the superstructure, on such an unprecedentedly deep base of rubble, might have been expected. The settlement amounted to about one-twentieth of the height of the mound, though it was not uniform. At the head the superstructure settled at least six feet. The nature of the settlement is illustrated by Fig. 224.

The head of the pier, Fig. 225, built in 1864, was designed with a view to obviate the defective features of the body of

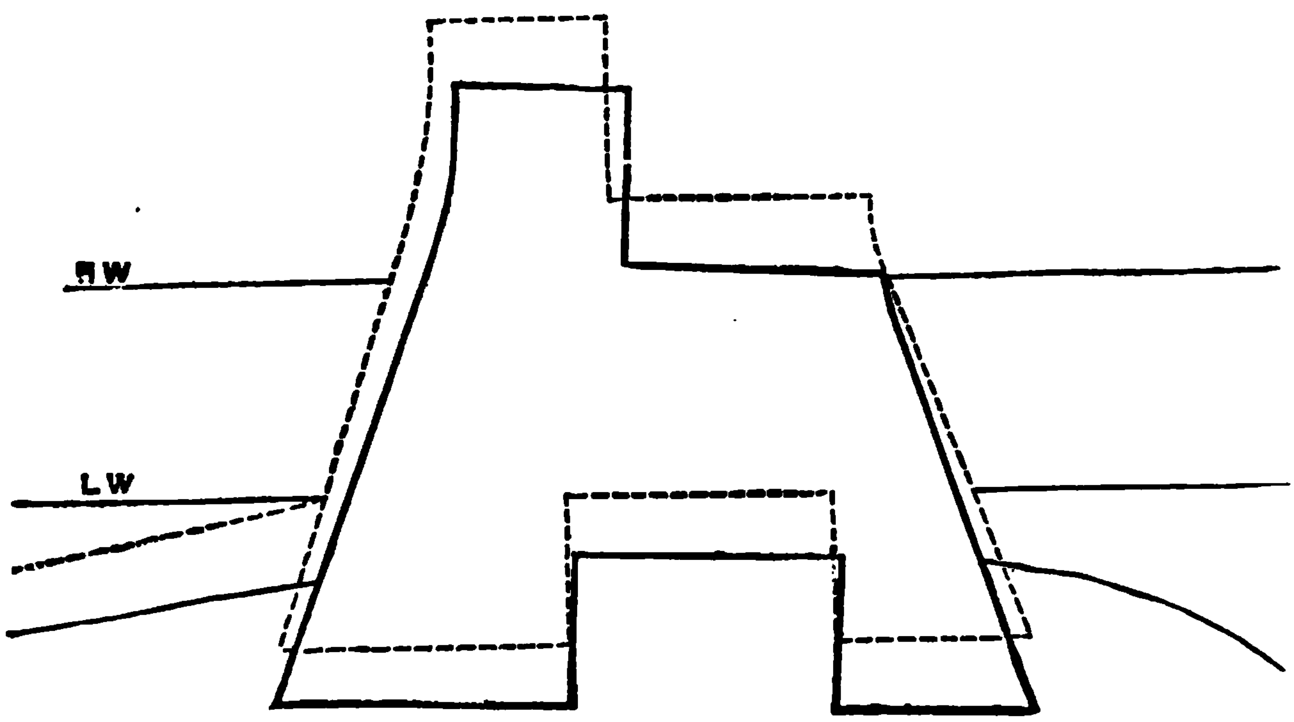


Fig. 224.—Alderney Breakwater: Settlement.

the structure, brought in evidence by the experience of years whilst the work advanced, and to bestow additional security on this exposed portion of the work. The foundations of the superstructure were carried down to a lower level than those of the preceding portions: for 22 lineal feet at 18 feet below low-water level; for 2 feet at 21 feet below; and for the terminal 42 feet at 24 feet below low water. The foundations were also carried solidly across the whole width of the breakwater—a width of 65 feet at the base—instead of

being constructed with separate walls, front and back. The sides were formed with a batter of 4 inches in 1 foot, terminating at a width of 31½ feet at the quay surface, which was finished 16 feet above high water. The end of the pier is finished square, a form which was adopted, in preference to the customary semicircle, for greater simplicity of workmanship, and to facilitate a junction for an extension at any future time. The facing of the sides and the end consists of nine courses of granite headers, 3 feet thick, of which the four uppermost courses were joggled and dowelled together. The backing, for 30 feet in height, consisted of blocks of

Fig 225.—Alderney Breakwater: Head of the Pier.

concrete in ten courses. The upper portion of the pier was built of Mannez stone in cement, carried up to the promenade level; the face-stones of the seven lowest courses being dowelled. Twelve courses of the corner quoins were further secured by iron bars and diagonal straps, bolted to the masonry. The rubble mound was finished against the faces of the head, at a level of 15 or 16 feet below low water, several feet lower than in the preceding portions of the pier. This portion of the work has stood unshaken on the mound. In 1872, eight years after it was built, it was stated that the head had cost nothing for maintenance, whilst the length of 1,400 feet immediately preceding had been the most expen-

sive to maintain; and it was estimated that at least 25,000 tons of large stone would be required to be deposited each year in order to maintain the sea-slope of the mound, and obviate the danger of undermining the superstructure, since the foreshore had already, in 1872, been removed by the action of the sea to a depth of 12 feet below low water in many places.*

The total cost of the works of construction and maintenance, extending over a period of twenty-five years, amounted to £1,274,200. Of this sum, £57,200 was expended in repairs of damages caused by the sea to finished and unfinished work; leaving the net cost, £1,217,000, at the rate of £259 per lineal foot, or £777 per lineal yard, for a depth averaging probably about 90 feet below low water.

Another instructive example of the characteristic behaviour of mixed rubble foundations of breakwaters at the surface of the sea, is supplied in the case of the breakwater at Holyhead Harbour. This breakwater consists, like that of Alderney, of a mound of mixed rubble quartz-rock from an adjoining hill, upon which is erected a substantial stone superstructure, finished with a transverse head, carrying a lighthouse, illustrated by the frontispiece. The breakwater is 7,860 feet in length, in a depth of water averaging 40 feet at low spring tides, with a maximum depth of 55 feet. Ordinary spring tides rise 17 feet; equinoctial tides, 20 feet. The slope of the foreshore, according to the section, Fig. 226, is 12 to 1 above low water, and for a foot or two below it, where it assumes a slope of 5 to 1, to a depth of 10 or 12 feet below low water, and thence about 2 to 1 to the bottom. Landwards, the slope is 1¼ to 1. The width of the mound at low water level is at least 250 feet. In 50 feet of water the width at the bottom is about 450 feet. The weight of the rubble

* See Mr. Vernon-Harcourt's paper on Alderney Harbour, in the *Proceedings of the Institution of Civil Engineers*, vol. xxxvii. p. 60.

stone in bulk is 1 ton per 20 cubic feet. The stone was deposited from a temporary wooden staging.

The rubble mound having been formed and consolidated by the action of the sea, the superstructure was erected, for the foundation of which the rubble was excavated to the level of low water. The principal object of the superstructure was to shelter the interior of the harbour and to prevent the loose deposit from washing into the harbour. The outer wall has a total thickness of 17¼ feet at the upper part, with a batter to the base which is 23 feet wide. The total height to the top of the parapet is 25½ feet above high water. The inner wall, distinct from the outer wall, is formed with a vertical face to the harbour, and is 8 feet

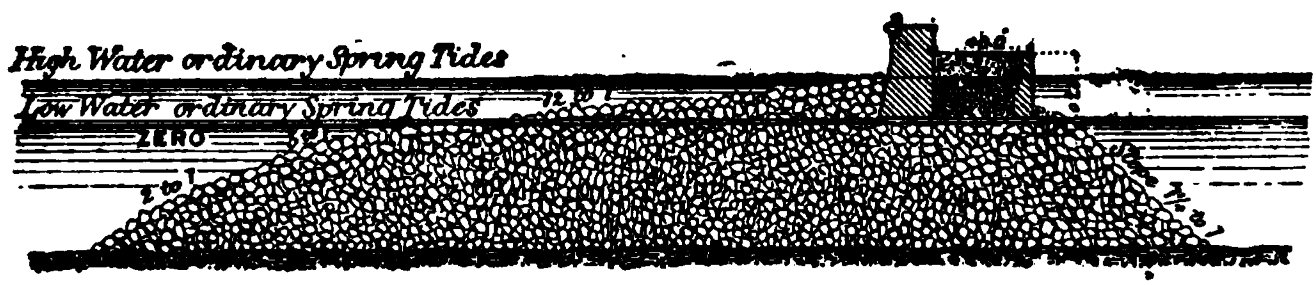


Fig. 226.—Holyhead Breakwater.

wide at the base. It stands 10 feet above high-water level. The width of the quay-way is 40 feet, and the total width at the base of the superstructure is 64 feet. The hearting consists of rubble. The total cost of the breakwater was £163 per lineal foot.*

The rubble on the seaward side is exposed to the force of the waves at all times of the tide, and it is stated that it is subject to shifting and drifting, and to be reduced to the condition of shingle. From the ascertained results of the action of the sea at Alderney, this effect is what might have been expected. The flatness of the exterior slope, 12 to 1, is no doubt much below the angle of settlement of rubble under the action of sea waves; and this consideration may have

* See Mr. Harrison Hayter's paper on Holyhead Harbour: *Proceedings of the Institution of Civil Engineers*, vol. xliv. p. 95.

formed an element in the original design, in the anticipation that therefore the rubble would not be disturbed. It is on record, nevertheless, that as a matter of fact the rubble foreshore has been in some places washed away from the lower part of the superstructure, and in other places piled up to within a few feet of the parapet.

Mr. David Stevenson records some of the results of wave-action on the superstructure of the breakwater at Wick, where it was exposed to the force of the waves at right angles to the line of its direction in about 30 feet of water. The outer or exposed part of the breakwater was founded on rubble at a level 18 feet below low water of spring tides, and was carried up to the level of 11 feet above high water, where it was 43 feet in breadth. The upper work was carried away during a storm, in which the waves were estimated at 42 feet depth from crest to hollow. Stones of 8 tons and 10 tons weight were carried over the parapet and lodged on the roadway of the breakwater. The experience gained at Wick proved beyond question that these waves did not affect the foundation of the walls at 18 feet below low water, all the damage having been confined to the superstructure, and extending to about 10 feet under low water. Below this level the work remained unharmed. As a protection against further damage to the end of the breakwater, it was resolved, in 1871, to construct a head by depositing cement blocks on the rubble base as a foundation for three courses of large flat stones, surmounted by a monolith of cement rubble built on the spot, as in Fig. 227. A course, *b b*, of 100-ton cement blocks was laid on the rubble; on these, two courses of 80-ton cement blocks, *a a*, were laid. On these, three courses of large flat stones were set in cement, surmounted by a monolith of cement rubble 26 feet by 45 feet long, by 11 feet deep, weighing, at 16 cubic feet per ton, about 800 tons. It was attached to the uppermost cement blocks by $3\frac{1}{2}$-inch iron rods, which passed through

holes cut in the flat stones, and were embedded in the concrete at each end. The whole of this united mass, indicated in the figure by tinting, weighing together 1,350 tons, was carried off bodily by the waves, slewed round by successive strokes, until it was finally removed and deposited at the innerside of the pier. The lower, or foundation course of 100-ton blocks, *b b*, laid on the rubble, remained unmoved; but the second course, *a a*, was swept off after having been relieved of the superincumbent weight. The head was restored

Fig. 227.—Wick Breakwater.

by the construction of a mass of united concrete masonry, weighing 2,600 tons. "Whether or not the billows, known locally as the 'wild rollers' of Wick Bay," said Mr. Stevenson in 1875, "would leave this mass of masonry undisturbed, remains to be seen." The huge mass did, as a fact, remain undisturbed for three years; but in January, 1877, it was carried bodily away, having been moved to within the line of the breakwater, where it lies in two pieces.

Breakwaters consisting of a combination of timber framing

and rubble stone have been constructed on flat sandy shores. To such situations alone they appear to be adapted, as they must be deficient in strength or durability for deep water exposed to the violent action of waves. Timber is a convenient material for making barriers against the sea if used in suitable situations. The Yarmouth pier is protected by the sands in the offing. Most of the Dutch piers, like those of Boulogne and Calais, and several on the English coast, are timber erections. But none of them are deep-water piers.

Two forms of breakwater of this class, Figs. 228 and 229, were adopted by Mr. Abernethy for the protection of the Port

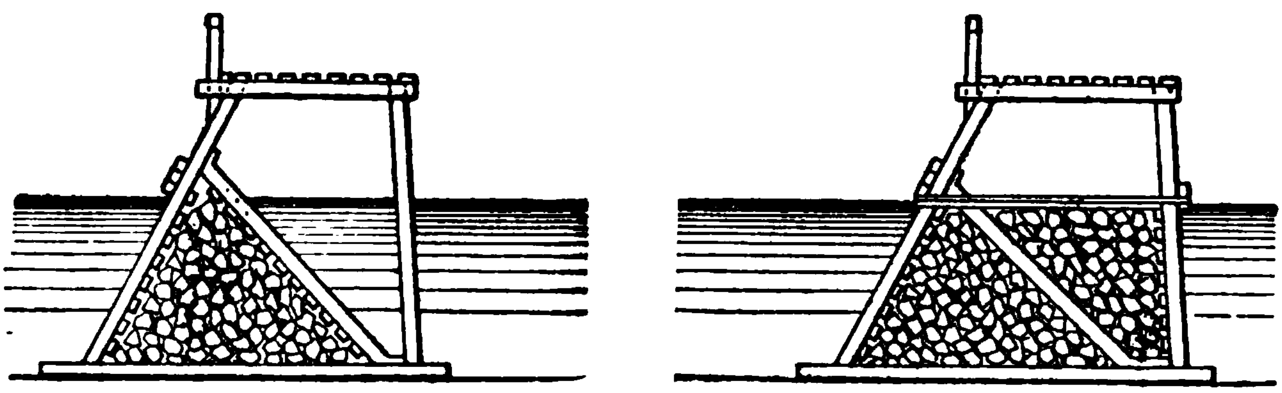


Fig. 228. Breakwaters with timber framing. Fig. 229.

of Blyth. Frames, consisting of a sole piece and two uprights, one of which is strutted with crossbearers, or half balks, to carry the roadway, are placed at intervals of 10 feet, and tied together longitudinally by walings and by open planking. The space thus comprehended is filled with rubble stone. Crossbearers at the upper part carry the roadway, also of planking. The second design, Fig. 229, was adopted for the further and more exposed portion of the breakwater. The timber was creosoted. At low water portions of the site are nearly dry, but there are 5 or 6 feet of water at the head. At high water, there is 22 feet depth of water. The cost, including that of the round end and the lighthouse, amounted to £11 per lineal foot.

Mr. D. Miller recommended a framed system of construc-

tion for piers and breakwaters. The framework is of iron, formed of piles or standards, and ties; and serves as the staging for all the constructive operations, and remains as an essential part of the work. It binds together a strong casing of stone or other sufficiently durable material, which encloses and forms the facing of the breakwater, the interior being filled with rubble or other cheap materials, which may be cemented into a solid mass by means of liquid concrete. This system was adopted in the construction of the docks and quay walls of the Albert Harbour at Greenock. By forming the walls under low water, by a combination of cast-iron piles and stone facings, slid down over and enclosing the piles, and of concrete backings, the work was commenced without the use of coffer-dams. The cost of the outer or sea piers, 1,200 feet long and 60 feet wide, was estimated at £63,000, being at the rate of £53 per lineal foot.

Crib-work, as it is called, is much practised in the construction of breakwaters at the lake harbours in America. Cribs, Fig. 230, are frames constructed of timber, from 30 feet to 50 feet in length, at least 20 feet wide even in the shallowest water, and never less in width than the total height from the foundation to the platform. The platform rises at least 5 feet above the high-water level of the lake. The bottom is grilled or grated, of timber. The timbers are 12 inches square, except for the lowest course, in which they are 12 inches by 18 inches. The cribs are built in still water to a height somewhat greater than the depth of water on the intended site of the pier. They are then towed to their places in succession on the line of the pier, from the shore outward, joining end to end. When a crib is in position, it is weighted with stone until it touches the bottom, when it is filled level with the top. When the cribs have settled down, the framing is raised to a height of 5 or 6 feet above high water, and filled up with stone. The bottom is dredged out to a level, when levelling is necessary, before

receiving the cribs. When the cribs have finally settled they are levelled up by the application of wedge pieces, when longitudinal planking is laid to form the roadway. It is usual to place a crib of larger size, 30 or 32 feet square, at the end of the breakwater. The behaviour of the cribs is

settle in the positions first taken up. The estimated cost

of constructing and sinking a crib 50 feet by 30 feet by 30 feet, in 24 feet of water, at Chicago, amounted to £31 10s. per lineal foot.

The elaborate network of framing employed in the inland waters of America for holding the rubble-stone hearting of breakwaters, contrasts forcibly with the widely arranged framing of European breakwaters of timber and stone, typified at Blyth Harbour:—indicating two things—plenty of timber, and a perception of the necessity for greatly subdividing loose material like rubble, in order thoroughly to fix it and hold it by open framework.

It has been seen that concrete manufactured into blocks of great size and weight has been, with great advantage, employed in the construction of the substructure of breakwaters and piers, in the capacity of *pierre-perdu* or in heaps. In such a capacity it serves in an admirable manner, by its saliencies and irregularities, to break up the masses of waves and to disperse and exhaust their energy. It has also been seen that blocks of concrete have been employed and laid in regular courses, in the construction of the superstructure of such works, where blocks of a magnitude and weight far exceeding those of the largest blocks of stone have been prepared and deposited.

The use of concrete in the construction of sea-walls was resorted to centuries ago, as may be seen at the ports of Genoa and Leghorn. The quay-wall was divided into compartments, the space was lined with canvas; and white lime, which was obtained in the district, was mixed with puzzuolana only,—no sand,—and passed through water. The first announcement in modern times of the method of building in water by concrete in bags, appears to have been made by James Frost, of Finchley, who, in his patent of the year 1822, states that "in constructing foundations, and walls, and piers, the parts of which may be under water, I mix the required cement with the before-mentioned hard

and durable substances; I enclose the composition in bags lowered by tackle to the surface of the work, and there dispose them in a regularly stratified manner, while the composition is in a soft state and will take the impression of the preceding layer in strata, and thus form courses of a well-embedded conglomerate rock."

Mr. P. J. Messent commenced using concrete in bags placed by divers under the foundation blocks of the walls of the North Pier, at Tynemouth, in 1865; and, in 1867, he used bags of concrete for the repair of the piers. But the employment of the system of depositing concrete in bags, in the general construction of works under water, appears to have been initiated contemporaneously by Mr. W. D. Cay, of Aberdeen, and Mr. J. Barton, at Greenore, in 1870.

Fig. 231. Breakwater, Aberdeen. Fig. 232.

The New South Breakwater at Aberdeen, Figs. 231, 232, in section, is an instructive example of Portland cement concrete work in water. It is 1,050 feet in length, in a depth of water of 22 feet 8 inches at the head, low-water spring tides. The total height at the head is 46 feet, and the breakwater stands 11 feet above high water, with a rise of tide of 12 feet 9 inches. At the section, Fig. 231, the [illegible] width at the top is 30 feet, and at the section, Fig. 2[illegible] it is 35 feet; the batter of the sides is 1½ inch to [illegible] in 1 foot. The foundations rest on granite rock, on boulders, and [illegible] and gravel, and on clay mixed with gravel. Upon th[illegible]mated c[illegible] ground.

which was cleared of loose stones and sand, a layer of bags of concrete was deposited. The bags of concrete, each holding 5 tons, were lowered in wrought-iron skips, the bottoms of which were on hinges, and were opened to let fall the bags when they were brought into position. Each bag was flattened out, and when it stood too high it was beaten down; or, if partially set, cut down. Small holes in the surface were filled with bags deposited by hand. The proportions of this concrete were, 1 of cement, 2½ of sand, and 3½ of gravel.

For a length of 363 feet, extending to low water and to the outer edge of the rocky foreshore, the breakwater was built of liquid concrete deposited in place in frames or cases. The upper portion of the breakwater, for a depth of 18 feet, was likewise constructed of liquid concrete to the head of the breakwater. Each piece of concrete, as laid, extended completely across the breakwater, and the lengths of pieces were from 8 feet to 31 feet, making pieces weighing from 335 tons to 1,300 tons. In the construction of the larger pieces, blocks of concrete were thrown into the mass. The concrete was composed of 4 of sand, and 5 of gravel, to 1 of cement.

From the bag-work in the foundations up to 1 foot above low water of neap tides, where the liquid concrete work just described was commenced, the work was composed of blocks of concrete 4 feet high and usually 6 feet wide, weighing from 10½ tons to 24 tons. The composition of these blocks was the same as that of the cement just described. Large rough pieces of broken stone were incorporated.

An apron of concrete was placed along the seaward side of the foundations, consisting of 15 bags of concrete, containing 100 tons each, to obviate the chance of damage from undermining by the sea.

The work was commenced in 1869, and completed, with a lighthouse, in 1873. The net total cost, including the

charge for plant and sea-staging, amounted to £68,000, being at the rate of £65 per lineal foot.*

For the proposed extension of the north pier at Aberdeen, shown in section, Fig. 233, the foundation is of sand, with solid ground at a depth of 7 feet. The portion under water up to 3 feet above low water was to be formed of bags of liquid concrete, each holding 50 tons. A wide platform of these bags was first to be laid as a foundation, and left to settle in the sand. On this platform bags were to be deposited so as to bring the surface above low water. On this

Fig. 233.—Breakwater, Aberdeen: Proposed Extension.

surface liquid concrete was to be deposited in frames in pieces of 700 tons each.

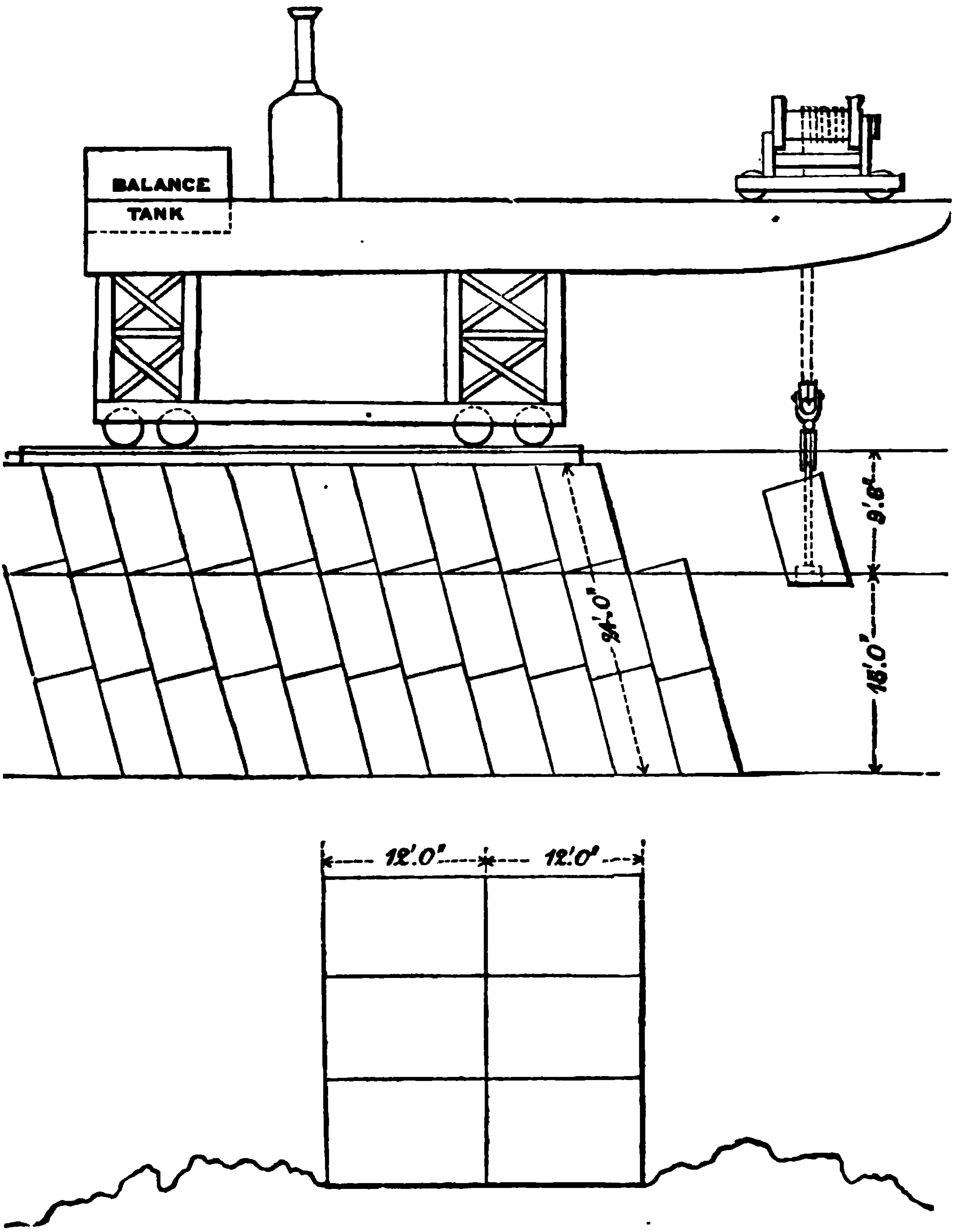
The apron, consisting of heavy masses of concrete in bags, forms an excellent protection for breakwaters in shallow water, where there is not depth enough for ordinary rubble stones to rest in security.

In the breakwater at Kurrachee, extending from Manora Point, on the west side of the entrance of the harbour, designed by Mr. W. Parkes, is to be found a novel mode of employing large blocks of concrete, superposed, in a longi-

* See Mr. W. D. Cay's paper on this breakwater in the *Proceedings of the Institution of Civil Engineers*, vol. xxxix. p. 126.

tudinally inclined position, for the construction of vertical-sided breakwaters. The breakwater is 1,503 feet in length, and terminates in a depth of 30 feet at low water. The style and construction of the breakwater are indicated in Fig. 234. The base is a bank of rubble stone laid on the natural bottom, levelled off for the most part to 15 feet below low water; but, near the shore, where the original depth is less than 15 feet, the bank is levelled to 10 feet below low water. It was found on trial that, in the shallow portions, where the rubble was heaped up to the level of low water, it was lowered by the waves to a depth of from 7 to 9 feet. The bank was formed to a width of 100 feet at the level of the foundation, two-thirds seawards and one-third towards the harbour. The superstructure was composed entirely of blocks of concrete, 16 cubic yards in bulk, weighing 27 tons, consisting of sand, shingle, quarry lumps, and Portland cement. The blocks were 12 feet by 8 feet, by 4½ feet thick. The attachment for lifting the blocks was made by means of two lewises passing vertically through the blocks. They were lifted by the "Goliath," a steam hydraulic travelling crane of 50 feet span; the traverse of the crane was 40 feet, and the lift was 3 feet 2 inches, worked by an 8 horse-power steam-engine. Each block was transported by a railway on a truck drawn by a tank locomotive to its place in the structure. The blocks were set by a crane called the "Titan," the end of which was overhung so as to carry the blocks of three tiers in advance to their places. The blocks were placed on end in an inclined position; they are three deep vertically, and, as shown in cross section, there are two independent rows of blocks side by side, making the width of the breakwater 24 feet. The lower ends of the lowermost blocks were bevilled so as to rest with level surfaces on the mound. The work was commenced in November, 1870, and completed in February, 1873. The total cost of the breakwater amounted to £93,565, being

at the rate of £62 per lineal foot, exclusive of general charges.



Figs. 234.—Manora Breakwater, Kurrachee.

The breakwater is constructed as two independent halves, divided longitudinally, and it might have been expected that

signs of weakness would be manifested, more particularly as the outer half rests on quicksand and the inner half on quicksand diversified by unyielding masses of rock. It has subsided as much as 3 feet into the sand, and at high water there is a depth of 4 feet of water over the breakwater, yet it is said to be perfectly effectual as a breakwater. Such a structure is scarcely fitted to withstand the shocks of heavy waves of translation. Its weakness has been demonstrated by the repeated removal of blocks during storms, and the necessity for binding together the blocks at the head by chain-ties. Though the seas at Kurrachee are not nearly so violent as on many parts of the coast of Great Britain, the concrete blocks appear to be scarcely sufficient without bond to withstand the force of the waves at Kurrachee. Nevertheless, by the absence of bond horizontally and the inclination of the blocks, a settlement of 3 feet was admitted without dislocation of the superstructure.*

Mr. Parkes has adopted a similar design for the construction of the breakwater piers of the harbour at Madras, in which there is from 24 feet to 42 feet of water. A base of rubble stone is formed up to a level 22 feet below low water. From this level to 3½ feet above high water, the superstructure is built of blocks of concrete, 27 tons weight, regularly placed by means of a Titan crane, in the same manner as was adopted by Mr. Parkes in the formation of the Manora Breakwater.

The extension of the South Jetty at Kustendjie in Turkey, designed by Mr. Liddell, was constructed of inclined blocks of concrete like the Kurrachee Beakwater, but each block extended across the jetty from side to side, and the blocks broke bond, as shown in Figs. 235. The original jetty, which had a length of 450 feet, was protected by a mole of *pierre-perdu* and blocks of concrete. The extension,

* See Mr. W. H. Price's paper on the Manora Breakwater, in the *Proceedings of the Institution of Civil Engineers*, vol. xliii. p. 1.

253½ feet long, was constructed, as a breakwater, of large blocks of concrete, weighing from 26 tons to 34 tons each, on a base of rubble stone. The blocks are in inclined tiers of four blocks, at an angle of 48°. By the inclination adopted, which was determined experimentally by means of a model, the centre of gravity of each block was thrown in advance of its base in the direction of the inclination, and so obviated any tendency of the blocks that might otherwise have existed to tip forward during settlement. The superstructure is 18 feet wide at the base, and 12 feet wide at the top, with a batter on each side. All the blocks are 6 feet high and 5 feet wide. The work is laid in 16 feet of water, and rests on 8 feet of rubble. The rubble is heaped on each side of the breakwater. The top of the work

Figs. 235.—South Jetty Extension, Kustendjie.

is 11 feet above water. The blocks were made of 1 part of cement to 2½ of sand, and 5½ of broken stone. When two or three days old, the top of each block was roughly dressed to a straight edge, to afford a fair seat for the one above it. The blocks were lowered when twelve or fourteen days old. The topping of the blocks, which brought the jetty up to the 11 feet level, was done in lengths of 28 feet, covering four tiers. The concrete for this purpose consisted of 6½ broken stone, 2½ sand, and 1 cement, and stones of from ½ to 1 cubic foot in volume. Each length of topping weighs 200 tons.

To prove the stability of the blocks alone, without any superincumbent load, five tiers were left untopped for a

whole winter exposed to heavy seas. None of them was disturbed or moved except by ordinary settlement. To secure the end of the jetty from excessive settlement, the soft bottom at the last seven tiers was removed by dredging to a depth of from 3 feet to 4 feet, and blocks were placed as a footing. The end of the jetty is simply rounded. The natural bottom is a mixture of sand and mud overlying stiff yellow clay, and the weight of concrete presses down the loose stone base. The blocks have, in every instance, settled vertically without disturbing the line of direction; the only effect of settlement having been to open the joints of the concrete cap, which has nowhere given way. The work was commenced in 1870, and finished in September, 1873.*

The two structures just described may fairly be contrasted, and it is easy to discern what should be avoided and what should be imitated.

Another like meritorious work was designed and executed by Sir John Hawkshaw, with upright walls—the North Sea piers for the Amsterdam Canal, Fig. 236. The piers were built of blocks of concrete, in horizontal courses, on a very unfavourable foundation consisting of quicksand. It was attempted at first to erect these piers by means of a timber staging on screw piles, but slight disturbances of the sea had the effect of excavating holes round the piles, laying them bare and deranging the staging. It was next attempted to deposit the blocks by means of a "Titan," after having partially excavated the sand, when the blocks should have made a bed for themselves, which could be levelled to receive the superstructure. This mode of procedure having proved also unsatisfactory, a layer of basalt rubble was thrown down, having a width of about three times that of the base of the pier. When the usual excavation by the action of the sea took place at the sides of the deposit, the

* See Mr. G. L. Roff's paper on the South Jetty, in the *Proceedings of the Institution of Civil Engineers*, vol. xxxix. p. 142.

stones dropped into the hollows until they found their natural slope, as indicated by the dotted lines in the figure, leaving stationary the central horizontal portion on which to build the pier. By this means a good and permanent foundation was obtained, and it was found as the work advanced that the trenches, which in places had been no less than 20 feet in depth, gradually filled up. The pier was built in courses of concrete blocks 8 feet thick, with a topping of concrete for the whole width. It is 32 feet wide at the base, and 22 feet wide at the top. It is 34 feet in total height, of which 13 feet is above low-water line. The rubble stone against the sides of the pier is 18 feet below low water.

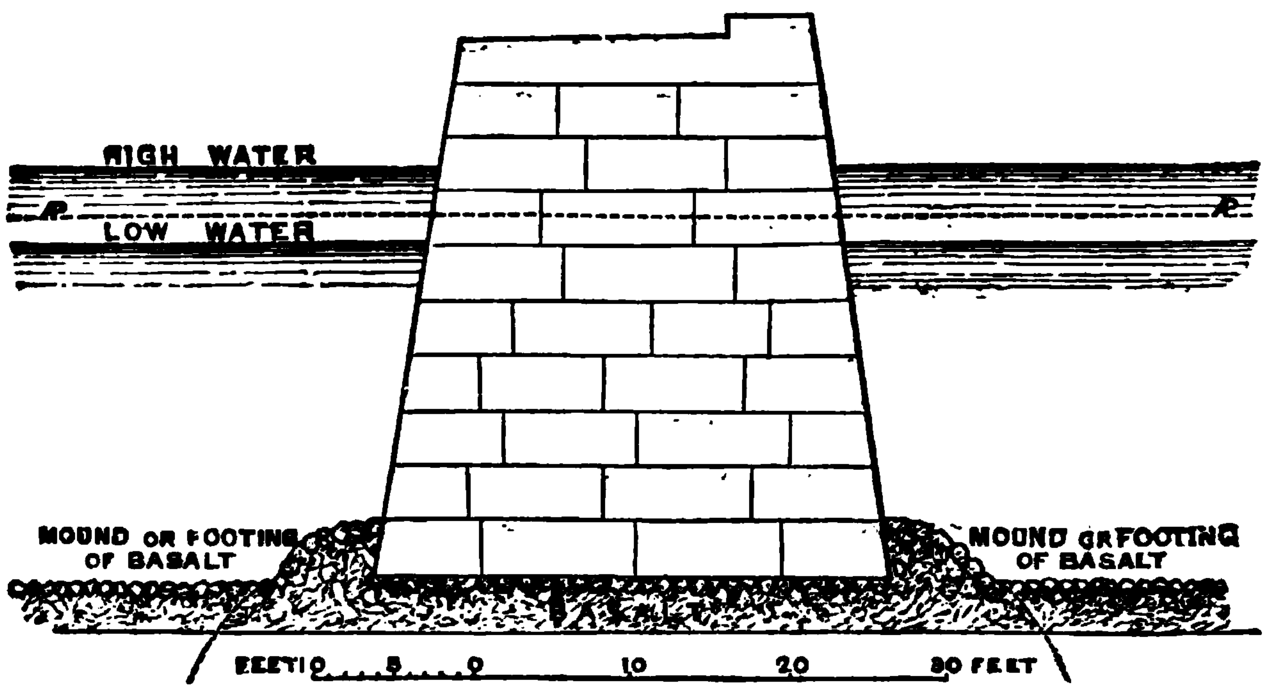


Fig. 236.—Amsterdam Canal: North Sea Piers.

Mr. B. B. Stoney, in 1874, advocated the application of his system of constructing large and heavy concrete blocks on shore and floating them to their destination, to be afterwards described, for the construction of deep-water piers and breakwaters with vertical walls. Comparing his system with the construction of the upright-sided pier at Dover, block by block, by means of diving-bells, he maintains that by his system the time as well as the cost of construction would be much diminished. "Where the depth at low water exceeds

from 30 to 40 feet, it may be desirable to use two blocks, one above the other, or to raise the foundation by a mound of rubble stone, extending to within 25 or 30 feet under low water; that is, to a depth below that at which the action of the waves is sufficient to move the stones forming the mound. A single, double, or triple row of blocks might be laid on the axis of such a mound after it has been allowed to consolidate, and even supposing that local settlement of the mound takes place, this will not affect the cohesion of individual blocks, nor will it in any way injure the adjoining blocks; and if a wide roadway is required on the top of the breakwater, rubble filling can be thrown in between a double row of blocks so as to obtain any desired width of pier." *

Whatever may be the ultimate depth at which sea-waves cease to be felt, it appears evident that they are practically harmless to rubble mounds at depths, varying according to the circumstances, of from 12 feet to 16 feet. And, as between sloped work and vertical walls in deep water, it is admitted that, in the case of a wave of oscillation, the upright wall receives the least amount of actual impact, and the only pressure is that due to the difference between the extreme height to which the wave rises on one side, and the depression formed by a similar wave on the other side. As a matter of observation and of theory, it is found that a pure wave of oscillation, whatever surface it impinges upon, is reflected from that surface at an angle corresponding to that at which it impinges. When the wave becomes a wave of translation, the upright wall founded at low water is the worst form that can be opposed to the wave, as it strikes the wall with the utmost accumulated force of the sea. An upright wall founded upon rubble work 12 feet below low-water mark, would, it

* See Mr. Stoney's paper "On the Construction of Harbour and Marine Works with Artificial Blocks of large size," in the *Proceedings of the Institution of Civil Engineers*, vol. xxxvii. p. 332.

is believed, be properly applied, as it is very unlikely that the wave would act upon the wall as a wave of translation. The proper alternative, if the force of the waves, as waves of translation, is to be exhausted before they reach the vertical wall, is to provide concrete masses of weight sufficient to resist unmoved the action of the waves, on a system similar to that which has been sketched by Mr. Murray, and adopted in the construction of breakwaters and piers abroad.*]

* See p. 386 *ante*.

CHAPTER VIII.

PIERS.

The disposition of piers, in plan, is a point upon which great diversity of opinion exists amongst engineers. For some reasons it appears desirable to construct them on two curved parallel lines, until near the extremity, with the convex side turned towards the direction of the progress of the alluvions. In this case the scouring action of the water from the inner harbour, whether produced by sluices or simply by the tidal action, will be more effectual against the bar which usually forms at the head of the inner pier, by the centrifugal force of the water deflected from the outer side. Moreover, this disposition is more favourable for the protection of the interior of the harbour from the effects of the wind.

But if the piers be executed in smooth-dressed masonry, the transmission of the waves takes place with undiminished intensity. The Romans appear to have noticed this effect, for all the old ports of the Mediterranean have a polygonal form, and it has evidently been an object with their engineers to avoid joining the several straight lines by curves filling in the angles. On the other hand, it was noticed in the port of Havre that the waves were reflected from the opposite faces of the piers, which were constructed of this polygonal form; and the manœuvres of the vessels were much impeded by the constant changes in the direction of the waves thus produced.

The roadway of the piers should be finished off at a height

sufficient to guarantee the men employed upon them from the effects of the sea in ordinary states of the weather. From 7 to 9 feet above high water spring tides will be sufficient; but the extremity, or the head, must be raised about 2 feet higher than the remaining portion. The heads, or extreme ends, may be erected with a width at the crown of from 27 to 36 or 40 feet; whilst the intermediate parts may vary from 7 to 20 feet, according to the materials employed. When the piers are in wood, the smaller dimensions are employed; stone piers are very rarely made less than 12 feet wide upon the line of the pavement.

The form to be given to the transverse sections of a pier is regulated by the nature of the materials employed in its construction, as much as by the dynamical effect of the waves. The materials may be either wood, loose rubble stone, or solid masonry bedded in mortar.

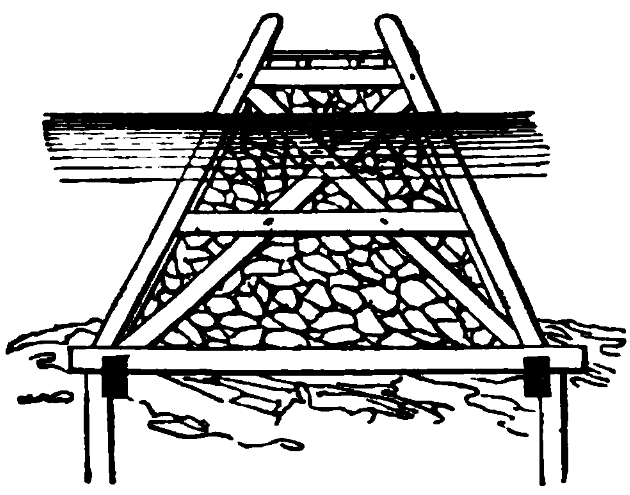


Fig. 237.—Wooden Pier.

Wooden piers may be entirely open, or filled in with rubble either entirely or partially; or occasionally they are placed upon the crown of a subsidiary pier, finishing at a point below the high-water line, which is executed in rubble masonry or loose stones. The lower part of the majority of wooden piers is, however, covered either by a mass of concrete, of loose stones, or of fascines, dressed with slopes both to the seaward and the inside of the harbour, forming a kind of ledge which serves to defend the foundations.

The frames of such piers are placed at distances apart of from 6 to 10 feet, according to the depth of water and the habitual agitation of the position in which they are to be placed. They are made in the form of a trapezium, the inclined sides being respectively turned towards the channel and the open sea, and forming with the vertical line an angle of from 13° to 33°. The timbers consist of two or three posts, tied together by horizontal clipping pieces, with raking struts or braces, forming with the horizontal ties a system of triangles. Wales, sills, and heads tie these separate frames together longitudinally. In the best works of this description, all the joints are made by halving and bolting, for it is found that the continual motion of the waves causes the tenons to work in the mortices wherever this style of joint is used, and that there is no effectual way of remedying the loosening thus produced; whilst, if the joints be halved and bolted, they may be tightened up by screwing the bolts, should they have worn. All the wood-work should be tarred, and precautions must be taken to defend it from the attacks of the boring worms, whose ravages will be noticed hereafter.

The upper sills carry joists, upon which is laid a planking, usually from 4 to 5 inches thick, and with spaces of about 1 to 1¼ inch wide between each plank, to allow of the escape of any water breaking over them. The planks are spiked down to the joists, and a species of bridging or tying-down joist is bolted upon their extremities to the sill resting immediately upon the framework.

When the piers are filled in with rubble stone, the cases to retain the latter are formed by close boards, laid horizontally against the upright posts. The interior is filled in sometimes with shingle, sand, or clay, as well as with stone, and the recent application of Portland cement concrete in the execution of such works appears likely to exercise important effects upon this branch of construction. The best position for the horizontal planking appears to be upon the outside

of the posts, because, although when it is placed upon the inside it resists the thrust of the materials inclosed more effectually, it will, on the contrary, when on the outside, destroy the action of the waves upon the framing to a greater extent. In the latter position also the planking can be more easily repaired, and no asperities are offered able to affect or be affected by any vessels which may rub against the jetties in passing. It is a very important rule to be observed in all constructions connected with piers or quays, that no essential parts of the framing be exposed to the abrasion of vessels either passing or stationary; wherever there is a possibility of any occurrence of this description, it is advisable to place a firring to protect the permanent work.

In modern piers the frames are made distinct from the piles or other portions of the woodwork in or near the ground. There is great difficulty, in fact, in driving with regularity such long piles as would be able to receive the flooring; and it is very easy to place the whole of one of the previously-prepared frames of the upper structure upon its foundations during the interval between two consecutive tides. The supposed advantage to the solidity of the structure in consequence of the posts being identical with the piles, it may also be observed, soon ceases to exist; for in a short time the destruction of the wood from the alternations of dryness and wetness, or from the attacks of the worm, render it necessary to replace portions of the work.

It has been observed that piles driven into the sea-shore are rapidly laid bare by the shock of the waves. The ground-swell acts upon the bed of the sea, and in time produces a conical depression round the head of the pile, whose depth may sometimes become as much as from 2 to 3 feet in a tide. These depressions extend on all sides, so that the piles are often laid bare for a considerable length; and to obviate this danger it is usual either to fill in between the pile-heads with concrete or with stone rubble, or, in Holland, to place a mat-

ting of fascines loaded with rubble, through which the piles are subsequently driven.

The advantages offered by wooden piers may be stated to consist in the fact that they are rapidly and economically constructed; the disadvantages they present consist in the frequency and cost of their repairs, and also that they do not effectually guarantee the interior of the harbour from littoral currents if totally or partially open, nor do they destroy the agitation of the waves. It is for this reason that in many ports the system of partially filling has been resorted to, and the practice usually followed is, to carry up the filling to the

Fig. 298. STONE PIERS. Fig. 299.

level of high spring tides in very exposed situations, or only to that of the high neap tides in others.

Stone piers are executed either with a hearting of rubble masonry or of concrete cased with ashlar, or with an embankment of earthwork cased by external walls tied together by cross walls, which form, in fact, so many separate compartments, or even occasionally of loose rubble. In the latter case, however, the inner face towards the passage of the harbour is executed in coursed masonry with a vertical or nearly vertical face, in order not to interfere more than is absolutely necessary with the water-way. Telford adopted innumerable varieties in the methods of bedding

and bonding the masonry for these various descriptions of piers, but there does not appear to exist any necessity for observing other rules in their construction than will be found enumerated hereafter.

In positions where it is easy to obtain large stones of a nature to resist the action of sea water, it is preferable to execute piers with ashlar facings, and, generally speaking, to fill in between them with rubble masonry or concrete. A vertical face, we have already seen, destroys the violence of the waves with greater rapidity and more effectually than a long slope, such as every loose rubble jetty must assume, provided that it be constructed of a sufficient dimension; and with the requisite conditions of bonding together the several parts, a wall of such a profile will require less repair than a mere heap of small materials, each of which is susceptible of being displaced in a storm. The largest stones which can be obtained ought to be used, especially at the height corresponding with the greatest agitation of the waves, and for the upper courses: horizontal bonding courses should be introduced at regular intervals, with plugs or dowels connecting the several stones, and the upper surface of the filling in must be carefully paved so as to throw off any rain or sea water falling upon it. The example given on the next page will illustrate the most theoretically perfect mode of executing such works; it is copied from the southern pier of the port of Havre, which is exposed to very violent storms in winter, and in all times to a powerful littoral current.

When the interior of the pier is filled with earthwork, as in Fig. 241, it is necessary to place the counterforts tying the walls together at distances varying from 16 to 45 feet in the clear, making the counterforts from 6 to 10 feet wide. The thickness of the retaining wall must exceed that absolutely required, to insure a resistance to the thrust of the embankment under ordinary circumstances, because the action of the

Fig. 240.—Stone Pier at Havre.

waves changes very materially their conditions of stability. The most important precaution to be taken is to pave the top so as effectually to remove any water falling upon it. The inclination to be given to the walls may vary from 1 in 4 to 1 in 8; perhaps the latter is preferable as a general rule.

In some positions the progress of the shingle is found to exercise an important and very destructive effect upon the

Fig. 241.—Pier filled with earthwork.

piers projecting within its line, in consequence of its friction upon the stonework. De Cessart recommended that a casing of planks from 3 to 4 inches thick be placed round the portions exposed to this abrasion, and that they should be nailed to horizontal walings let into the stonework. If the use of such boarding be objected to, it will be necessary to execute the portion thus rubbed by the shingle in granite, and under any cirumstances to avoid the use of soft argillaceous or calcareous stones.

The foundations of jetties should be executed in the most substantial manner possible, and either upon a general bed of concrete, or upon a platform laid upon piles and surrounded by sheet piling; if the subsoil be of a nature easily removed by the repercussion of the waves or the action of the current, it may also be necessary to construct a wide apron, in a similar manner to the one executed at Havre, represented in the Fig. at page 421. These aprons are more peculiarly

Fig. 242.—Jetty at Honfleur.

required at the head of the jetties, where the ground swell is the greatest; and they should be carried down as low as possible, the upper surface having either a slope or a curvature towards the open sea, so as to decompose the shock of any waves breaking on it.

If the extremity of the pier be carried out far into the sea, so that the foundations be below the lowest tides, it will be found almost impossible to execute them in a coffer-dam. The modes hitherto employed under such circumstances have been to construct them of loose rubble stone up to the low-water mark, as in the cases of the piers at Aberdeen erected by Telford, or of those at Honfleur; or to

execute that portion with concrete, or with masonry sunk in caissons.

The loose rubble foundations answer in positions where there is no danger of the substrata on which they repose being removed, and when the passing current carries a sufficient quantity of mud or sand to fill up the interstices of the stones. Works of this description, if executed in tolerably deep water, assume the profile upon a line of direction of the prevailing wind, which may be thus described :—On the outside, and in the part situated below the usual action of the waves, the slope of the materials, as they arrange themselves naturally, will be tolerably steep, and about 1½ to 2. In the zone exposed to the action of the waves it becomes about 6 to 11; and the inner slope, which is of course protected by the other, assumes the proportion of 1 to 2. The lower slope, or the one beneath the action of the waves, is, however, the only one which has any fixity, so to speak; and it is also to be observed, that in shallow water the ground-swell appears to destroy it even in this portion, and to give rise to constant changes in the outline. For instance, a rubble jetty executed upon a ledge, called the Boyard, in the roads of Aix, on the western shores of France, was continually undermined at the foot, although the foundations were placed at 14 feet be'ow low-water line. It is therefore necessary to cover the smaller stones, of which the body of such jetties is composed, with blocks of considerable dimensions at the feet, and also in the zones exposed to the action of the waves. The immediate position of the intended jetty should also be covered entirely by a bed of concrete, executed after a sufficient time has been given to allow the subsidence of the rubble to take its full effect, and the masonry elevated upon this concrete. The sketch on the preceding page, representing the jetty of Honfleur, at the mouth of the Seine, will illustrate this construction.

When the oundations are executed in concrete, the ex-

ternal edges should be protected by sheet-piling, and precautions must be taken to prevent their being undermined. The surface of the place intended to receive the concrete must be cleared of any alluvial mud or peat, and as far as possible of any compressible substratum. But the most important condition for insuring the permanence of this description of work is, that the lime or cement used be of a nature able to resist the chemical effects of the sea water.

Smeaton executed the foundations of the Ramsgate Harbour in caissons, sunk afterwards upon beds previously prepared to receive them by means of the diving-bell. The piers are carried out about 300 feet upon a chalky bottom, at a depth varying from 8 to 10 feet below low-water mark of spring tides. The caissons were about 10 feet wide and 34 feet long, measured perpendicularly to the axis of the pier. The heavy storms which blow upon this part of the coast from the S.E. moved the caissons, until they were weighted by the superincumbent masonry.

Fig. 243.—Pier at Kingstown Harbour.

Jetties entirely in loose rubble work are principally constructed for the purpose of destroying the force of the waves without its being intended to make them serve for the purpose of assisting the manœuvres of vessels entering or departing. Such a mode of construc-

tion may be advisable when the rough materials are easily procured, and when skilled labour is exorbitantly dear; but, as a general rule, it will be found that the ultimate expense of the maintenance of such works will more than counterbalance any economy in the original outlay. Telford has, however, executed several jetties in this manner, of which an illustration, Fig. 248, from the eastern arm of the Kingston Harbour, is selected. In other cases, as at Peterhead, he made the inner side with a vertical face of dressed masonry; but it is to be remarked that the occasions on which that eminent engineer resorted to this mode of construction were decidedly exceptions from his usual course, and that wherever it was economically possible to execute jetties in coursed masonry he resorted to that system.

In addition to the interference with the translation of waves into the interior of harbours offered by the form and disposition of the jetties, or by the breakwaters before described, that object is sometimes effected by means of spurs projecting from the inner face of the jetties into the harbours. This is a system only to be resorted to upon extraordinary occasions; for, as the spurs project into the direct line of the channel, vessels entering the harbour must occasionally be forced to go through manœuvres attended with considerable danger. The practice of engineers, of late years, has certainly been to avoid any kind of deviation from the regularity of outline of harbours for this reason. The submersible jetties, formerly constructed in prolongation of the principal ones, or more frequently of the leeward jetties, have also been abandoned. Indeed it was found that, as they were covered at high tide, they became little else than sunken reefs in the course of vessels entering, and gave rise frequently to serious complications of the tides and currents of the port.

[The distinction between piers and breakwaters is rather

shadowy. A pier carried into deep water becomes a breakwater. A breakwater joined to the shore, and formed with a roadway or a promenade on the top, becomes a pier.

Several illustrations of pier-work have been produced in the preceding discussion on breakwaters. It remains to notice a few characteristic piers, designed specially as piers, though at the same time acting designedly or incidentally as breakwaters.

The most conspicuous instance of a pier, which also operates as a breakwater, and furthermore as a groin in arresting the flow of shingle on the south coast, is the pier at Dover, Fig. 244, which has been constructed with nearly

Fig. 244.—Pier at Dover.

upright sides from the bottom. The choice of this form was, it is said, induced by the want of suitable stone in the district. The material below low water was put in place with the aid of diving apparatus, and the work was of course expensive, costing £290 per lineal foot for the first contract of 800 lineal feet; and for the second contract, let in 1854, £415 per lineal foot. It is constructed with granite facings and a breasting of rectangular blocks of Portland cement and shingle concrete up to a little above half-tide level; above this the filling consisted of liquid concrete. The pier, though

at present it is essentially a landing-pier, is intended ultimately to be extended so as to form a harbour of refuge. It is formed with a uniform batter on each side, and is in section about 80 feet wide at the base and 42 feet at the top, comprising a roadway 30 feet wide, and a heavy parapet on the seaward side. It is founded 45 feet below low-water mark.

The south or west pier at Whitehaven, Fig. 245, designed by Sir John Rennie, and constructed in 1834, was built on a foundation of sand. The pier was built of the soft sandstone of the district. It is 61 feet wide on the platform. The outer wall, or sea wall, has a curved face battering one-half its

Fig. 245.—West Pier, Whitehaven.

height. It is 18 feet thick at the base, and 14 feet at the top. The inner, or quay wall, is curved inside and outside, with a batter of one-fifth of its height. It is 8 feet thick at the bottom, and 7 feet at the top. Both walls are strengthened by counterforts. The total width across the top amounts to 75 feet, and at the base to 100 feet. The parapet of masonry on the top is 12 feet high, 10 feet thick at the bottom, and 7 feet thick under the coping. The coping is 11 feet wide, formed of large stones, resting on a broad cavetto moulding, curving over both on the inside and the outside. The top is slightly rounded, and forms a promenade fully 8 feet wide.

The walls are constructed of ashlar masonry filled in with

rubble, solidly bedded and grouted together. The roadway was paved with large blocks of ashlar for a depth of 3 feet, and bedded in hydraulic mortar. The parapet is composed of finely dressed stones, dowelled and bonded vertically and horizontally. By means of the peculiar form of the exterior and interior of the parapet, the waves, in heavy weather, are in a great degree reflected and prevented from breaking over the wall; so that, except in extraordinary gales, scarcely any water is thrown on to the pier. Even then, the inner projecting coping affords a shelter.

The base of the outer wall is protected by an apron of large stones, bedded in the sand, and covered with a mass of large rubble up to the level of low water. The rise of spring tides is 25 feet.

Fig. 246.—New North Pier, Whitehaven.

The north pier was designed like the south pier, but of smaller dimensions.

The new north pier, Fig. 246, a spur of the old north pier, separating the outer harbour from the north harbour, was constructed a few years since. It was a part of other works constructed to the designs of Mr. Brunlees for the improvement of the harbour. It is 50 feet wide at the quay level. The outer wall, next the outer harbour, is 16 feet wide at the base, on a foundation of concrete 19 feet wide and 3 feet deep. The wall is double, strengthened by cross walls, and filled, in the "pockets," with concrete. The

wall is built of red sandstone, faced with ashlar block-in-course, with a batter of 1 in 12, and carries a parapet. The inner wall, against which vessels lie, is similarly constructed, but the pier is of hammer-dressed rubble laid in broken courses, or "snecked." The coping is of sandstone ashlar, 8 feet wide and 18 inches thick, dowelled at the joints and grouted with cement. The filling is of rubble, and the roadway is ballasted, and carries two lines of railway.

The contrast between this recently designed pier and the older south or west pier, Fig. 245, is to be remarked. The leading features in contrast are the inclinations of the walls; of which the total batter in the older pier is 1 in 2, and in the recent pier only 1 in 12. The relative efficiencies of these forms respectively are marked by the fact that, during an on-shore gale, very little sea rises over the vertical wall, whilst the other piers are enveloped in spray.

A pier of a very different character, the new north pier at Sunderland harbour, was constructed by Mr. J. Murray in 1843. The harbour wall is of ashlar masonry of freestone, curved and battered like that at Whitehaven, and built on piles. It is 6 feet thick at the top and about 11 feet at the base, with counterforts. The foundation was laid 5 feet 4 inches below low water; the rise of the spring tide is 14½ feet, and the top of the wall stands 10 feet 9 inches above high water; the top is paved for a width of nearly 40 feet, divided into two parts, of which one is raised 2 feet above the other, with a parapet wall 2 feet 3 inches thick. A rubble backing, 18 feet wide, next the parapet on the seaward side, led to a long glacis of ashlar pitching to a slope of 4½ to 1, making the total width of the pier, from the face of the harbour wall to the footing of the glacis, about 250 feet. The action of heavy seas on the pitched glacis was such that receding waves tore out the stones, which had been previously loosened by the impact of the advancing waves,

or had been blown up by the air between the joints subjected to hydrostatic pressure.

By experience of three years of the north breakwater, Mr. Murray was induced to construct the south pier, in 1846, with a glacis to the seaward side, entirely of loose rubble stone, leaving the action of the sea to adjust the slope. It was finished at a lower level than that of the north pier,

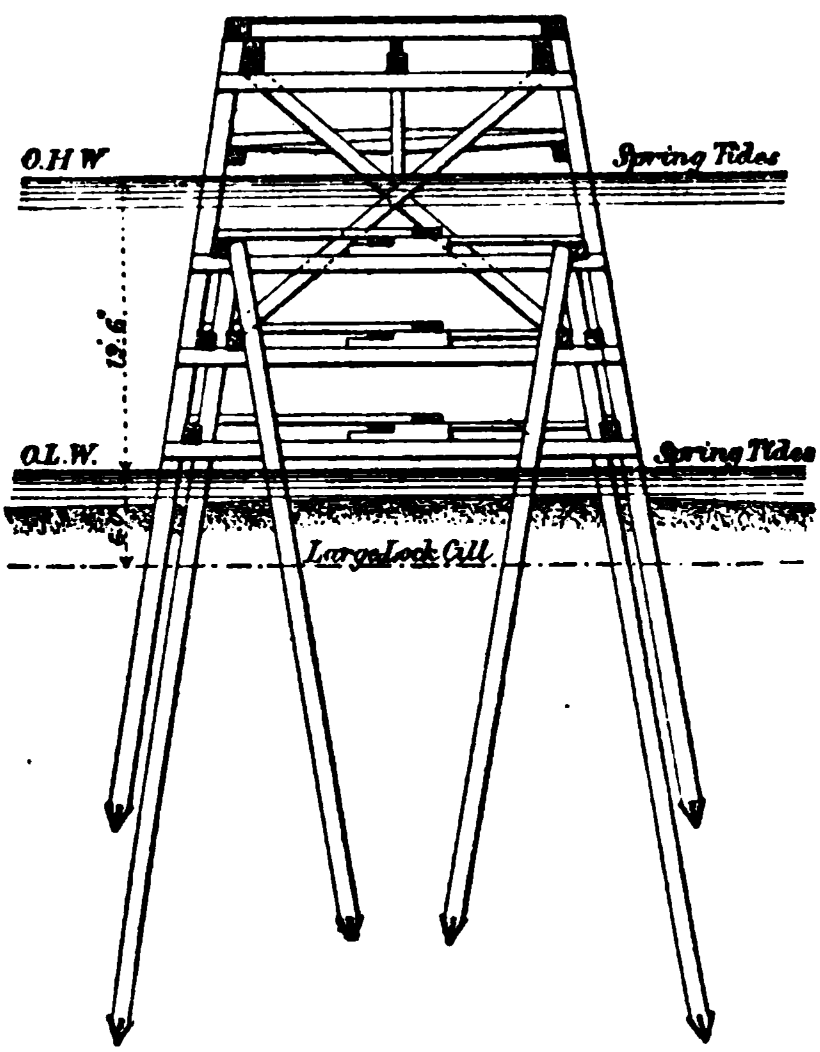


Fig. 247.—Timber Pier, Great Grimsby Docks.

with a level berm about 8 feet above high water, and a slope of 6 to 1. By this depression of the level of the backing, a substantial sea wall was demanded for the back of the pier as well as for the harbour side. This pier is less exposed to the action of the sea than the north pier.*

A simple timber pier is represented by Fig. 247. Two piers of this form are erected at Great Grimsby docks,

* See Mr. Murray's paper on "Sunderland Harbour," in the *Proceedings of the Institution of Civil Engineers*, vol. vi. p. 256.

bounding the tidal basin. The construction is of open timber, in bays of piles in clusters, 25 feet apart. The front piles average 50 feet in length and 14 inches square, with a batter of ¼ inch to 1 foot. The interior or abutting piles average 36 feet long and 14 inches square. Each pile carries a wrought-iron shoe of 28 lbs. weight. The piles are supported and shored by walings and diagonal braces. The outside of the piers, facing the sea, is protected by half-timber fender-pieces, spiked to the walings, which answer the double purpose of keeping the heavy seas from injuring the lock-gates,

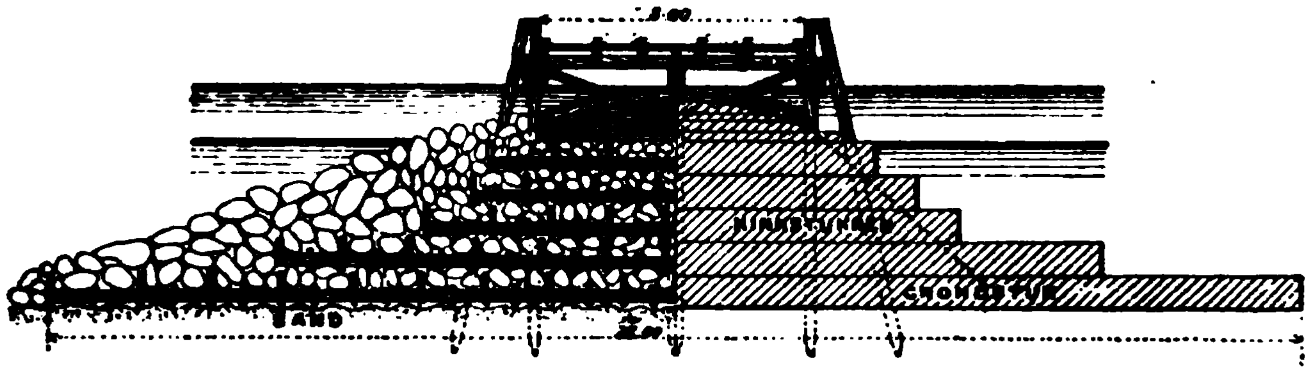


Fig. 248.—South Pier, Rotterdam.

and of checking the rush of the tides through the basin. The planking for the floor is laid with ¾-inch spaces, and wrought-iron mooring rings are fixed at intervals of 100 feet along the inside of the piers. The whole of the timber was creosoted.

Dutch engineers, since the year 1850, have made trial piers on the system of fascine embankments in vogue in Holland. Two piers or moles have been in course of construction, since 1863, to form an approach to Rotterdam. They are formed chiefly of fascines. The south pier, Fig. 248, 1,258 yards in length, is finished, and ends in 16 feet 5 inches of water, at low water. The northern pier was, in 1875, completed to a length of 2,078 yards from the shore. The piers are constructed of successive layers of "zinkstukken," or fascine mattresses, 15 inches thick, weighted with 10 cwt. of stone per square yard. For the body of the pier, there are required from 5 to 6 mattresses, averaging, with the stones,

39 inches thick. They are further held in place by five rows of piles driven about 11 or 12 feet through this mass into the sand below. The outer slopes and edges of the mattresses are covered with a coating of stone averaging 350 cubic feet per lineal foot of pier. The part above water is covered with larger stones retained by rows of small oak piles, the ends of which project above the level of the work with the view of breaking the force of the waves. The crown of the southern pier is 26¼ feet wide, rounded on the upper surface, which attains the level of ordinary high water. The base of the work is 124⅓ feet wide. The piles connecting the mattresses are carried to a height of 9 feet 10 inches above the top of the piers. A roadway is fixed to the piles, carrying lines of rails for the conveyance of materials for the construction of the pier. The height of the pier is from 13 feet to 16½ feet. The cost of the south pier was £145,000, being about £38 9s. per lineal foot. The theory of this kind of breakwater-pier is that—1st. As it is to a certain degree elastic, the shocks from waves produce less injury to it than they would do upon a rigid and divided mass. 2nd. That, in a short time, the internal interstices will be completely closed with sand, whilst the exposed surface will be coated, and, so to speak, agglomerated with sea shells, weed, &c., so as to become eventually a solid mass.*

A pier, simply and solely for the purposes of landing and embarking, may be constructed very differently from the solid and substantial structures which have been described. In principle it may be made so that it shall not at all interfere with the movements of the sea—erected on piles. One recent example of open pier-work is supplied by the pier constructed at the seaport of Huelva, in Spain, for the Rio Tinto Mining Company, to the designs of Mr. G. B. Bruce.

* See Mr. T. C. Watson's paper on "The Use of Fascines in Public Works of Holland," in the *Proceedings of the Institution of Civil Engineers*, vol. xli. p. 158.

The design of this pier supplies an excellent instance of opportunities seized and applied in the arrangements for working the railway traffic on the pier by gravitation. The pier, Fig. 249, is constructed on cast-iron screw-piles and columns, braced together with wrought-iron struts, stays,

Fig. 249.—Pier, Rio Tinto Mining Company.

and tie-rods. The piles are screw-piles arranged in groups of eight piles, in two rows of four. At the shore-end their distances apart, transversely, are 7½ feet, 12 feet, and 7½ feet successively. Longitudinally they are 15 feet apart between centres. In the deep-water portion, the piles are all 12 feet

apart transversely. The screw-piles forming the base of the pier are 16 inches in diameter and 1¼ inches thick. The screw-blade on the lower end of each pile, Fig. 250, is 5 feet in diameter, with a pitch of 6 inches, and is held in place by two bolts, though it takes its bearing on a collar cast on the piles. The screws were cast separately from the piles, for convenience of transit from England to Spain. In case any of the piles should work out of position during the operation of fixing them, a radial joint of 4 feet radius was arranged at the surface of the ground at the shore-end, and in the deep-water portion at low-water level, for adjustment of the upper lengths of the piles. The piles were screwed to depths varying from 15 feet to 32 feet below the surface of the ground. They were turned by means of a capstan-head fixed on the upper end, with eight arms of from 8 feet to 15 feet in length, worked at the shore-end by 16 men, though in deep water from 45 to 110 men were employed, where two capstans worked from two stagings were attached to the pile. The screws descended by amounts varying according to the nature of the soil traversed. Whilst the pitch of the screws was 6 inches, they descended, for one turn, 4½, 5, 6, 8, or 9 inches, according as the soil was solid or light. It was necessary at some places to clear each pile of its core, and to loosen the sand into which it was screwed. This was effected mostly by forcing water down through the piles.

Fig. 250.—Screw-pile.

It was proved by repeated trials, at the shore-end, that the ground was incapable of supporting more than 700 lbs. per superficial foot. The bearing of the piles was, therefore, supplemented by that of timber platforms composed of 12-inch square balks, placed round each group of piles trans-

versely to the centre line of the pier and supporting the piles, which were fitted with cast-iron discs for the purpose of taking a bearing, on sills laid longitudinally. The discs were fixed in place on the piles so as to take their bearing on the sills after the platforms had settled down under a proof-load. The bearing area of each platform and the group of screws amounted together to about 1,000 square feet at the shore-end, and 1,500 feet in deep water, so that the gross loads supported per square foot of bearing surface were respectively 691 lbs. and 718 lbs. per square foot.

The columns which rest on the screw-piles are 15 inches in diameter, of cast iron, 1 inch thick, upon which caps and girder-beds are cast. The columns are 15 feet from centre to centre longitudinally, and are made up in lengths with external flanges. The flanges are faced and bolted together, making one continuous length from the screw to the cap on the column. The piles and columns constituting a bay are strutted and braced together by longitudinal and cross girders, and horizontally by channel irons. Diagonal tie-rods $1\frac{1}{4}$ to $1\frac{3}{4}$ inches in diameter, and angle-iron stays, are applied. The main lattice-girders over the 15-feet openings are 4 feet deep. There are three floors or stages for lines of rail at three levels. The roadway of the first floor is supported by cross-bearers of pitch pine, 11 inches square at 5-feet centres, riveted to two vertical angle-irons forming a pocket. On these the longitudinal bearers for the rails, 12 inches by 9 inches deep, are laid. The roadways of the second floor are carried on corbels.

There are four sets of spouts for the shipment of ores, two on each side of the pier head. They are constructed to meet the varying levels in the rise and fall of the tide, and the different heights in vessels. Each set of spouts has four fixed divisions. The shoot is raised or lowered by side chains working in sheaves on a cross-bar spindle under the inner end, and is adjusted to angle of $1\frac{3}{4}$ to 1. A steeper angle

than this admits of a too rapid descent of the ore. At a less inclination, the ore does not readily clear itself from the spout. The quadrant and pinion, with hand-gear, fitted to the derrick frame for moving the spout horizontally over the ship's hold, are most useful in trimming the ship during the operation of loading. The flow of the mineral from the shoot is regulated by a door at its lower end, as is usually done in the shipment of coal. The door is controlled by means of two side chains, one on each side of the shoot, worked from two oak drums fixed on one spindle working in two carriages in the derrick frame.]

CHAPTER IX.

QUAY WALLS.—DOCK WALLS.

The quay walls of a harbour are required to fulfil the same conditions as the walls of river wharfs, that is to say, to resist the lateral thrust of the ground, and to facilitate the discharge of vessels. For the attainment of the latter condition, it is important that they be as nearly vertical as possible; but in proportion as this object is attained, the stability of the wall itself is diminished. The late estimable Mr. Rennie, and after him the majority of English engineers, endeavoured to reconcile the two conditions by building their quay walls with a curvilinear batter on both sides, laying the courses normally to the curve as in Fig. 251. By this means the stability of the wall for the same cubical quantity of masonry is certainly increased, and the face stones—being, in fact, voussoirs—connect the masonry intimately throughout. At the same time the mechanical difficulty of execution is greater than in a wall with horizontal masonry, and this mode of construction entails the necessity of using inclined piles, which should be avoided as much as possible. The Dutch engineers occasionally incline the two faces of the wall in a direction parallel one to the other, and so that they overhang on the inner side at the top. The radiating joints in such cases are dispensed with; but the stability of such walls, especially when constructed upon soft mud, is never satisfactory, for the overhanging portion of the masonry adds to the lateral thrust of the earth upon that portion of the foundations and

wall below the centre of gravity. Walls of this description may almost always be noticed to have yielded at the feet. In the French ports upon the Channel it is customary to build the walls nearly vertical in the manner represented in Fig. 252,

Fig. 251.—Quay Wall. Fig. 252.—Quay Wall, Havre.

which is taken from the quay wall of the outer harbour at Havre. The mean thickness adopted in these ports is not less than 0·40 of the total height considered as unity; and, strange as it may appear to our ideas upon the subject, it is

found practically that this great thickness is not sufficient to insure the stability of the walls in many cases, for many of them have yielded.

The rounded outline of the bottom of ships admits of the formation of a set-off, or of an apron, to protect the foundations. In the case of the docks at Antwerp, as the rock in which they were excavated was sufficiently solid to dispense with the necessity of a facing of masonry, the set-off was formed in the rock itself. This is, however, a course of proceeding which should only be resorted to when there can be no danger of the undermining of the walls.

The thickness to be given to the quay walls, and the precise mode of construction to be adopted, must evidently, from what has been said above, depend upon local considerations of the cost of materials and of labour. The most important theoretical consideration affecting them is to be found in the fact that the earth behind them is exposed to be alternately wet and dry twice a day, and that the capillary action of the ground causes this action to rise to a greater height than the limits of the tidal range. The earth in this condition must be considered to be a semifluid mass assuming naturally a slope forming an acute angle with the horizontal line. But the most serious difficulty attending the construction of the quay walls of ports arises from the yielding of the mud under the foundations. If the mud lie upon a solid substratum which can be reached by piles, it is possible to found the wall in such a manner as to guarantee it from any danger arising exclusively from the vertical pressure. But it frequently happens that the mud moves laterally under the compression of the earthwork behind the walls, driving out their foundations, and forcing up the bed of the harbour. Accidents of this description occurred at Southampton, Lorient, and Rochefort; and it appears that if the stratum of mud be of great thickness, the only effectual mode of combating the danger is to lighten the vertical pressure of the filling behind

the walls by means of fascines, timber platforms, or by hollow vaulting. The quay at Lorient is erected upon a bed of mud of unfathomable depth, and in this case both the wall and the platform behind it are carried upon piles driven with the broad end downwards.

Guard piles ought to be placed in front of quay walls to protect them from the abrasion of the vessels moored alongside as they rise and fall with the tide, or from the shocks of vessels driven against the walls, occasionally with considerable violence. These piles need not descend below low-water mark of neap tides; they are usually bolted to the masonry, and covered with an iron cap. In the angles of harbours, staircases or inclined roads may be placed, to assist in unloading small boats. The only important precautions to be observed in their formation are, that all external arrises be rounded off, and every description of projection likely to injure the bottoms of vessels studiously avoided. The same remarks apply to all ladders, mooring rings, or other facilities for the manœuvres of the port.

[The lock walls of the Victoria (London) Docks afford an instance of iron framework and masses of concrete in combination, similar to that employed in the construction of the Brunswick Wharf, Blackwall. The piles are arranged in bays 37 feet 8 inches in length, 7 feet 1 inch from centre to centre of the main piles. The intervening space is occupied for a depth of 15 feet from the top by three cast-iron plates, retained laterally by the edges of the main piles. The lower intervening space is occupied by cast-iron sheet piles 20 feet long, three in each bay. In the rear of each main pile, 18 feet distant from it, a timber land-tie 20 feet long is driven. It is connected by tie-bolts to the main pile, and the interspace is occupied by concrete walling.

To Mr. B. B. Stoney must be assigned the merit of developing to the fullest extent the capacities of the system of con-

crete work for the construction of large structures in the sea. In 1861 he designed and proceeded with the execution of quay walls forming a portion of a large tidal basin and other works at the port of Dublin, with artificial blocks of large size, containing nearly 5,000 cubic feet of material, weighing 350 tons. The blocks, when laid in place, reach from a depth of 24 feet below equinoctial low water to 3 feet above that level, being 27 feet high. They are 21 feet 4 inches wide at the base, and 12 feet long in the direction of the wall; and, when set in place, a length of 12 lineal feet of the quay is laid, at one operation, up to ordinary low-water level. The upper portion of the wall, 15 feet 10 inches in height above low-water level, is built of concrete in the usual manner by tidal work, and is faced with granite ashlar to offer a smooth bearing for ships. It is coped with granite in blocks of from 2 tons to 4 tons. The total height of the wall is 42 feet 10 inches. Vertical grooves, 3 feet wide and 18 inches deep, are made in the sides of each block, so as to form a well 3 feet square between every two blocks. This well is filled with concrete which acts as a dowel, and effectually closes up the blocks. The concrete consists of 1 part of Portland cement to 7 of harbour ballast. The outer face of each block is formed of calp limestone quarried near Dublin, having smooth joints, easily squared. The form for the sides and back of the block is moulded in planking, and the hearting consists of rough stones weighing from 2 tons downwards, about which the concrete is packed by means of tamping irons. The blocks were manufactured on a side stage, and were removed and placed by means of floating shears constructed for the purpose. The cost of the quay wall so constructed, 43 feet high, amounts to £34 per lineal foot, or to £40 per foot including £6 for interest on plant. Mr. Stoney estimates that the cost, if built in the ordinary manner by coffer-dam, would be more than twice this sum.

Contrasting with Mr. Stoney's monolithic system, the con-

struction of Mr. Brunlees' dock walls at Avonmouth, Fig. 253, may be referred to. The walls were built in a trench, on

Fig 253.—Dock Wall, Avonmouth.

each side of which piles were driven. Stretchers were introduced as the excavated material was removed. The founda-

tions were of blue lias concrete laid to a depth generally of 6 feet below the level of the floor of the docks, and 4 feet thick. At other places, where the ground was a weak clay, the foundation of concrete was carried down to the sand to various depths down to 17 feet below the floor, as indicated by dot lines in the figure. On the blue lias concrete, 2 feet below the floor of the dock, the wall was built of Portland cement concrete mixed with rough blocks, faced with ashlar work in Pennant stone, to a height of 18 feet above the floor. The concrete was tipped into the excavation from the surface, out of barrows, and at the back it was rammed against the piles and poling boards. The front of the wall was carried up in ashlar, averaging 2 feet in thickness, at the same rate at which the cement concrete was put in. The upper part of the wall was backed with coursed rubble of the ordinary description. By building the lower part of the wall in concrete, and the upper in rubble, time was saved as well as cost. The concrete set more quickly than rubble, and in ground easily affected by wet weather and liable to settlement that was a matter of great importance. The total height of the wall above the foundation is 42 feet. The thickness at the base is 17 feet, and that at the top is 7 feet. The foundation is 20 feet wide.

The quay walls of the Albert Docks, Hull, opened in 1869, were constructed of sandstone masonry. The foreshore near the quay is covered with a deposit of Humber silt, or, as it is locally called, warp, in some places 30 feet thick. This deposit thins out towards the water-line, and is succeeded by a bed of peat of from 2 feet to 8 feet thick. Beneath the peat there are two beds of clay, separated by a bed of sand. The western wall, Fig. 254, was founded in the sand at a depth of 3 feet below the bottom of the dock, on a bed of concrete, defended by sheet piling in front. On the north side, where the sand thinned out, the masonry was placed on the clay direct, without concrete or sheet piling. The

section adopted for the south wall, after a failure of the bank had taken place there, are shown in Fig. 255. For the west end, where the sand extended to 19 feet below the bottom of the dock, two rows of close sheet piles were driven 20 feet 9 inches apart, the back row being 25 feet long, and the front row 15 feet long. The sand was excavated to a depth of 8 feet below the bottom of the dock, and was replaced by concrete up to a level of 4 feet below the bottom. At other places, where the wall rests direct on clay, sheet piling is only driven at the back of the foundation into the sand.

Fig. 254.—Dock Wall, Hull. Fig. 255.—Dock Wall, Hull.

The foundations of the walls of Junction Dock, at Hull, opened in 1829, were laid entirely on piles driven into loose soil, as sketched in Fig. 26, page 44. The modern practice of distributed foundations on concrete, in substitution for troublesome and costly rows of piles, as exemplified in the most recent dockwork at Hull, affords a pleasing contrast.

In the construction of the Marseilles docks, Fig. 256, the quay walls were built of blocks of concrete on a rubble base. At a depth of 19 feet 8 inches below low water a rubble embankment is formed, having a base about 28 feet in width, with slopes of 2 to 1. On this foundation a wall, consisting of four courses of blocks of concrete, is built. Each course is

about 5 feet deep, and the four courses make a total height of 19 feet 8 inches to the water level. These courses were loaded with two loose courses of similar blocks for six months in order to settle the bank; after which they were removed, and the wall was built up about 8 feet more, to the level of the quay. The width of these blocks is about 11 feet. The bank of rubble is carried up behind the wall to the level of the upper blocks of concrete. In several places the embankment beneath the wall moved by slipping forward and causing the walls to lean over into the docks. The heap of stones piled up on the back of the wall appears to be well adapted for thrusting the wall outwards.

Fig. 256.—Quay Wall, Marseilles Docks.

The quality of a dock wall is of little importance compared with the quantity. It must have weight to enable it to resist the chafing and bumping of large vessels, and it should be sufficiently strong not only to hold any amount of any kind of backing laid against it, but to carry a head of water equal to its height if it were left dry on the other side. The section, as a section, may be amply strong, and yet there may be a fault in the foundation. Mr. Alfred Giles mentions a dock wall erected by him, which had a width of 25 feet at the base resting on gravel, and a height of forty feet above the bottom of the tideway to the coping. That wall moved, though not seriously, but enough to show that a base of 25 feet was not sufficient for a wall of that height. Founded on gravel 5 or 6 feet deep, the wall did not move; but when

there was only 3 feet of gravel on a substratum of clay the clay moved, and the wall slid forward some inches. The walls of locks, fortified by inverts, do not slide. The inference is that if a mass of concrete or other heavy deposit were laid in front of the wall at the base the wall would not move; or if a wall be well founded on a wide base of rubble, like the wall at Marseilles, to provide a non-slipping foundation, wanting the pile of rubble backing thus applied, the conditions of stability would be met.

This argument leads to the consideration of two illustrative instances in point. Mr. James Barton's quay at Greenore, Fig. 257, has a total height of 47 feet 6 inches; it is 15 feet wide at the base and 7 feet 9 inches wide at the top, with a batter of 1 in 9. It is founded 21½ feet below low water; there is a rise of tide of 16 feet, and the top of the wall stands 10 feet above high water. The bottom generally consists of sand and gravel, but there is a short length of rock. The ground was dredged to a depth of about 4½ feet, and the bottom was levelled by divers as the work proceeded. The wall was faced with concrete blocks, weighing 3½ tons, laid header and stretcher. The blocks were 3½ feet deep for the two lowermost courses, and 3 feet deep upwards; the width of the wall was alternately 4 feet 2 inches and 2 feet 10 inches. The back of the wall was built of soft concrete in bags, each containing a cubic yard; the use of which was initiated in September, 1870. The bags were dropped from skips through the bottom, which was opened for the purpose, and adjusted by divers. The bags fitted to each other and to the hearting of soft concrete which was deposited from skips without bags. At the toe of the wall, a protecting body of blue clay puddle is bedded, 2½ feet deep, to prevent the escape of sand, covered by a layer of rubble stones 2 feet deep, extending 15 feet in width from the face of the wall. Mr. Barton considers that the face of the wall might very well have been constructed of bags of concrete,

like the back, instead of regular blocks of concrete, which demanded exact adjustment.

Section of Quay Wall, showing Bags

Fig. 257.—Quay Wall, Greenore.

The last section of the quay wall, 200 feet long, including the return wall which faced the littoral current, was, under low water, formed of blocks of concrete, weighing 100 tons each, as shown in Fig. 258. They were formed in moulds constructed of 3-inch planks, just above low water, and their size was regulated by the fact that they were to be lifted by flotation before high water. Each block was of the entire thickness, and was 10 feet long in the face of the wall. The cost of the quay wall, 840 feet long, averaging 45 feet in height, including fenders, excluding the dredging of the site, was at the rate of £27 8s. per lineal yard.

Section of Quay Wall, showing Blocks

Fig. 258.—Quay Wall, Greenore.

With respect to the use of bags of concrete for submarine construction, there is no difficulty in effecting a good jointing between the bags. They are easily adjusted by divers so as to be packed together, and make

very solid work. A curious illustration occurred on the coast of Holland, where a sailing vessel laden with bags of Portland cement foundered close to the coast. When, two or three weeks later, the bags were recovered and brought on shore, the cement in every bag had become solid, and the solid had assumed the precise form of the bag, even to the weft and woof. It had to be broken up and used as macadam. The water, of course, had penetrated into the body of the cement.

The second illustrative instance of a quay wall stably

Fig. 259.—Quay Wall, St. Louis Canal.

founded is the quay wall, Fig. 259, at the mouth of the Rhone, facing the entrance to the St. Louis Canal. The wall, 11 feet thick, is composed of four blocks of concrete, each 11 feet long, 8 feet 7 inches wide, and 4 feet 1 inch deep, on a base of rubble 6½ feet deep and 26 feet wide. The concrete wall was provisionally loaded with tiers of blocks before the superstructure was added, a precaution generally practised by French engineers to prevent undue settlement. The wall is surmounted by a superstructure of rubble faced with ashlar

8¼ feet high and 9½ feet wide. The cost of the wall amounted to £48 per lineal yard, including dredging, staging, and loading.

From the foregoing discussions, it is apparent that, in many situations, foundations are laid and constructed without the aid of coffer-dams. The French engineers resorted to the system introduced by M. Vicat so long since as 1813. This system consists in forming enclosures of sheet-piling up to low-water level and filling in with hydraulic concrete, on which the piers are built in the usual manner. Messrs. Bell and Miller, it has been noticed,* constructed, about the year 1862, sea walls and quays in deep water without the aid of coffer-dams: forming the walls under low water of a combination of cast-iron guide-piles in the front, with a continuous stone facing, which is slid down and over the piles, enclosing them, and of concrete backing deposited in a soft state.

Mr. T. E. Harrison, in 1870, adopted the plan of building a quay wall, where there was 30 feet depth of water at low water, upon cylinders, and arches between the cylinders, with sheet-piling of cast iron at the back.

Mr. J. F. Bateman adopted the system of brick-cylinder foundations for the Plantation Quay at Glasgow, where there was a depth of from 50 to 80 feet of quicksand. A coffer-dam to get down to the rock was out of the question. The cylinders he employed were 12 feet in diameter externally, built in a succession of rings of brick and Portland cement, the rings being each 2½ feet deep and 2¼ feet in width of rim. The cylinders are grooved and tongued into each other, and are straight and vertical. The average depth to which they were sunk is 52 feet 4 inches below the surface. Each ring of which the cylinder was composed was 9 tons in weight, and the rings were placed one above the other to weight the cylinder and send it down. When extra weight

* *Ante*, p. 401.

was required, iron rings 5 inches thick and of 5 or 6 tons weight were added. The excavation was accomplished by means of Mr. Milroy's excavator.

Wharves or quays of timber are exemplified in Fig. 260. It forms the face of a part of the embankment near the

Fig. 260.—Timber Wharf, Albert Docks, Hull.

Albert Docks, Hull, fronting the Humber. It is 16 feet wide, and the floor stands 5 feet above high water. It is constructed on two rows of piles, with sheet-piling behind the front row. The back row of piles is tied back to a third row.]

CHAPTER X.

DOCKS.

[A DOCK is a receptacle for ships. There are *wet docks*, in which the ships float at all times of the tide, for loading cargo or unloading it. They have been called "floating docks" also, but "wet docks" is the better designation. There are *dry docks*, *graving docks*, or *repairing docks*, for the cleaning and the repair of ships. There are also *floating dry docks*, commonly called *floating docks*, an expression now employed to signify a dock which floats, and carries a ship in it or upon it. Here it may be noted that the expression "floating dock" has drifted from its original signification.

The port of Liverpool is now the chief port in the empire. The power to construct the first dock in Liverpool was given by the Dock Act of 1709. Subsequently it was known as the Old Dock, and the level of its sill was adopted, and has been used ever since, as the established datum of the port. It was a very humble beginning, not more than 3½ acres in extent; but it is worthy of observation, from the fact that it is one of the first recorded instances of the application of gates to a dock for the retention of water within it at all times at a nearly uniform level, to complete it as a wet dock. To the engineer of the dock, Mr. Thomas Steers, is due the honour of being one of the first to introduce dock gates. During the first hundred years from the opening of the Old Dock, little progress was made at Liverpool in dock works. In 1816, the total wet-dock area was only 34 acres. In

1825, nine years later, it was extended to 47 acres; in 1846, to 108 acres; in 1861, to 220 acres. The new works now in course of construction at the northern and southern ends of the Dock estate comprise, at the north end, a half-tide dock of 18 acres, having two lock entrances, with an open basin direct from the Mersey; a great dock with three branches leading out of it, of an aggregate area of 43 acres, and specially intended for the use of the large ocean-going steamship trade; an 18-acre dock for the accommodation of the coal trade; a repairing dock of 8 acres; and 2 graving docks. The new docks at the south end will include special provision for the local carrying, import, and other trades, of 32 acres. The total additions, north and south, will amount to 114 acres of wet-dock area, with nearly 6 miles of quayage. On the completion of these great works, the area of the wet docks in Liverpool will amount to 365 acres, with 24 miles of quayage. The Birkenhead docks exhibit a total of 160 acres, with 9 miles of quayage. Taken together, the total wet-dock area in the estate of the Mersey Docks and Harbour Board will amount to 525 acres, or upwards of four-fifths of a square mile, with a lineal quay frontage of 33 miles. The general plan of the docks at Liverpool and Birkenhead is exhibited in Fig. 261; and a table of dimensions and proportions of the Liverpool docks is added, page 456.*

* The Editor is indebted for this historical information about Liverpool Docks to the Address of Mr. J. F. Bateman, President of the Institution of Civil Engineers, published in the *Proceedings*, vol. lii. p. 15; for the general plan to Mr. J. N. Shoolbred's paper on the "River Mersey and its Estuary," in the *Proceedings*, vol. xlvi. p. 21; and for the tabulated particulars to Mr. J. B. Redman's Chatham Lectures on Marine Engineering.

Fig. 261.—Liverpool Docks.

Fig. 261.—Liverpool Docks—*continued from the top of page 454.*

DOCKS AT LIVERPOOL, 1876.

Name of Dock.	Ratio—length to breadth.			Length.	Breadth.	Area.	Length of Quayage.	Length per acre.	Depth of water, high-water springs.	
				Feet.	Feet.	Acres.	Feet.	Feet.	Ft.	Ins.
Brunswick	3	to	1	1,260	420	12	3,360	280	25	4
Queen's	4	„	1	1,380	336	11	3,432	312	24	10
King's	2	„	1	840	420	8	2,520	315	23	10
Albert	1⅜	„	1	621	450	6⅓	2,142	338	25	2
George's	2	„	1	690	345	5	2,070	414	23	4
Prince's	5	„	1	1,500	300	10	3,600	360	24	9
Trafalgar	2⅝	„	1	780	300	5⅓	2,160	405	25	5
Clarence	1¾	„	1	750	405	7	2,310	330	22	0
Bramley Moore ..	2⅝	„	1	1,050	400	9⅝	2,900	301	24	10
London	2¼	„	1	960	420	9¼	2,760	298	25	4
Huskisson	4	„	1	1,580	378	29	7,836	270	..	
Branch No. 1 ..				1,260	240					
„ No. 2 ..				1,200	300					
Canada	2	„	1	1,200	600	16½	3,600	218	25	4
Docks Extension.										
Upper South End, Herculaneum, and Extension	1⅝	„	1	750	440	7½	2,230	297	..	
Branch to ditto....	4⅜	„	1	660	150	2¼	1,470	653	..	
Combined	..			..	..	9¾	3,700	379	..	
Proposed dock below	4⅜	„	1	1,330	300	9	3,260	362	..	
Proposed Lower Dock	3⅞	„	1	1,420	375	12¼	3,590	293	..	
Proposed North Brunswick ..	18⅝	„	1	1,120	60	1½	2,360	1,573	..	
LowerNorthEnd, proposed Half-tide Dock	1⅝	„	1	1,200	700	19¼	3,800	197	..	
Branch Repairing Dock	5⅝	„	1	840	150	3	1,980	660	..	
Wet Dock	3	„	1	1,500	500	17	3,100	182	..	
South Branch	4⅓	„	1	1.300	300	9	2,900	322	..	
Centre	4⅝	„	1	1,380	300	9½	3,060	322	..	
North	3⅞	„	1	1,150	300	8	2,600	325	..	
Sum of the last four Docks ..	..			..	..	43½	11,660	268	..	
Jetties to Branch Docks	..			500	..	..	14,660	337	..	
Mineral Wet Dock	3¼	„	1	1,600	500	18½	4,200	228	..	

The form of the enclosed area of a wet dock is governed by the conditions of the site. The square dock is the least costly, requiring the shortest length of wharfing for a given area. But it presents of course the minimum length of quay accommodation for the area, which is a disadvantage. The oblong rectangle is more serviceable. The earliest docks in the metropolis—the West India Docks—present the proportion of length to breadth as 5 or 6 to 1, and they have the reputation of being the most convenient docks in the river.

The entrances to docks are amongst the earliest subjects for consideration in the design of docks, especially on a rapid tideway. Respecting the best form and direction for the entrance, the entrance should be placed at such an angle to the course of the tide that ships may be docked with facility—an acute angle pointing to the flood, that is, pointing upwards. But, for undocking a vessel, she is brought out head first; and if the entrance point up stream, she would be liable to kant in the wrong direction. A direction nearly square to the stream is recommended as the best for wet docks; or an acute angle of 60° with the flood. The direction is of less importance if there be made a great width at the entrance, by rounding or splaying the wings.

The next question affecting the approaches is the design of the lock, whether it should be single or double; that is, whether it should be provided with two pairs of gates or three pairs, and whether there should be an outer half-tide basin for the accommodation of vessels entering and departing in one tide, so that the operation of docking and undocking should not affect the level of water in the docks.

For the construction of the dock, the excavation is to be considered, and the formation of the quay banks beyond the dock walls; also the preliminary borings to ascertain the nature of the strata below the lowest work; pumping water from the excavation whilst in progress; coffer-dams, and the

most economical mode of raising the excavated material to the surface; the condition of the floor as to porosity or lightness; the character of the quay walls and the backing; entrance locks, and bridges across them; warehouse and shed accommodation.

Steam-power is used for raising the material excavated, by winding engines or by locomotives. For them, of course, lines of contractors' rails are laid. Accurate borings must be taken at both extremities of the proposed site and at intervals along the quay walls, and at entrances or connecting locks. They should be carried down below the level of any foundation work. It is possible, nevertheless, to overbore the site. Over-boring may happen where competitive designs or estimates are in preparation, when the bore-holes may not be properly plugged up. A celebrated instance of this kind occurred at Hull in the construction of the Albert Dock. A dock had been proposed there for thirty years prior to the actual commencement of the dock, and borings had been made at various times. Now the alluvial soil of the Humber consists of a bed of silt 30 feet thick, on a seam of stiff clay and pebbles from 30 to 40 feet thick, succeeded by a stratum of quicksand of equal depth overlying the chalk. The foundations for the dock were in some parts carried down into the sand, and one of those old borings, made as far back as 1838 by Mr. Stead, of Hull, had been carried down more than 120 feet, to the chalk. When the excavations were in progress an enormous quantity of water and sand boiled up through bore-holes, and gave so much trouble as to cause the removal of the site for the apron of the entrance lock to be shifted out of their way.

Quay walls have already been treated. The manner in which quay walls are backed is a point of importance. The slope of the excavations behind the walls should be benched in horizontal steps, and the backing behind the walls should

be laid in thin courses well rammed. When gravel is excavated a backing of thin concrete relieves the walls very much of undue pressure from freshly deposited backing material.

To prevent access of tidal water under the aprons of locks, where arched inverts have been dispensed with, it has been customary to drive rows of sheet-piling at the outer and inner margins of the apron, and across the apron at the pointing sill, down to the solid substratum. Thus the passage laterally of subterranean water is prevented. But many locks and docks are now built without any sheet-piling, on a mass of concrete stepped down lower at the pointing sills, to effect the same object as that of sheet-piling, namely, to do away with a straight horizontal joint under the work, and to cut off the water, and also to support the weight of the gates.

The application of hydraulic power for moving gates and swing-bridges at docks is very generally resorted to. But it is not to be taken as of universal application, for it would be quite inexpedient in a dock where the trade is small and the opening of the gates infrequent, or in the case of a swing-bridge that is opened only once or twice a day. Where there is a large traffic, it is a great advantage to be able to open the gates in a minute and a half, especially in spring tides, when the tides remain only for a short time on a level; and frequently, from there being a pressure of shipping, it is of the greatest importance to work the lock to the latest moment. Another useful application of hydraulic power is to work capstans on pier-heads. A capstan may require thirty or forty men to work it; the confusion and labour are avoided by the substitution of hydraulic machinery.

An excellent typical example of docks designed and constructed according to modern practice is well described and illustrated by Mr. L. F. Vernon Harcourt—the New South Dock in the Isle of Dogs, forming part of the West India

Fig. 262.—New South Dock, West India Docks.

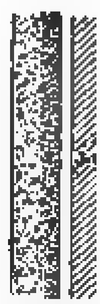
Docks.* From this account the following notice and illustrations are derived.

The New South Dock, on the Thames, was constructed according to the designs of Sir John Hawkshaw. The works (Fig. 262) consist of a dock having an area of 26¾ acres, joined to a basin of 5¼ acres, by a passage with reverse gates. The west entrance to the City Canal, on the site of which the dock was constructed, is retained; but at the eastern extremity a new entrance has been made, leading from the river to the basin. On the south side of the dock, foundations for five warehouses have been built and the superstructure erected (1872) on three of them, with a quay shed in front.

The main dock is 2,650 feet in length, and 450 feet broad; having quay walls all round it. The bottom of the dock is 29 feet below Trinity high-water mark, and

* See Mr. Vernon Harcourt's paper in the *Proceedings of the Institution of Civil Engineers*, vol. xxxiv. p. 157.

the surface of the quay is 5 feet 10½ inches above this level; making a total height of 34 feet 10½ inches. On the north side of the dock there are sixteen jetties, affording accommodation for 32 vessels. Down the centre of the dock a line of buoys, one opposite each jetty, is laid for the purpose of mooring the vessels. The dock walls, Figs. 263 and 264, are of brick, built hollow, and filled with



Vertical Section of Dock Wall. *Horizontal Section at A.B. and Plan.*

Fig. 263. New South Dock Wall. Fig. 264.

concrete. They are built on a foundation of concrete, varied in thickness according to the nature of the soil. The total thickness of the wall at the base is 18 feet. The front, the back, and the cross walls are of brick, bonded with hoop-iron. The front is 3½ feet thick, the cross walls 2 feet 4 inches thick, and 10 feet apart; and the back 14 inches thick, its function being chiefly to keep the concrete filling in place until it becomes consolidated. The face-work is built of best picked London stock-bricks, and the rest of the brickwork of ordinary stock-bricks. The wall is coped throughout with Bramley Fall Stone, 18 inches thick and 3 feet wide, dowelled with slate dowels, 4 inches

square, run in with Portland cement. Lias lime was used in the concrete and the mortar up to a level of 6 feet above the foundations. For the remainder of the wall greystone limestone was employed. The batter of the walls is 1 in 12. The floor of the dock is covered throughout with a layer of puddle 18 inches thick.

The jetties are 130 feet long and 25 feet wide, and are of Baltic timber. They are formed of upright timber fixed to longitudinal sills, strutted diagonally, with longitudinal joists at the top, covered with 4-inch planks. The sides and ends of the jetties are protected with vertical fenders of American white rock-elm.

As the sills of the old lock of the City Canal, at the west entrance, are 6 feet higher than the bottom of the dock, a row of sheet-piling was driven across the end of the old lock where it joins the dock, to prevent the sills being undermined. The ground was sloped from the top of the sheet-piling, at the level of the bottom of the lock, down to the bottom of the dock, protected by Bramley Fall pitching.

The dock is separated from the basin by a passage, 176 feet long and 55 feet wide, which being constructed with two pairs of gates, reversed, admits of the level of the water in the dock or in the basin to be raised or lowered independently of each other—an object of importance for the satisfactory working of the two entrances, for high water does not occur at both ends at the same time. The foundations of the walls and the floor of the passage are of concrete, carried to a depth of 13 feet below the bottom, under the sills and walls of the dock, and in line with the sills for a width of 17 feet. The gate floors and aprons are on a level with the bottom of the dock and basin; and the level of the sills is 2 feet higher, or 27 feet below Trinity high-water mark. The floor of the passage is a pavement of ashlar masonry, 2½ feet thick at the gates and 2 feet thick elsewhere, except at each end, where a course of headers and

stretchers, 3 feet thick, is carried right across. The walls of the passage are of solid brickwork, with the exception of the portion forming the foundation of the swing-bridge, over the centre of the passage, where concrete has been introduced. The heel-post stones, the hollow quoins, and the sills are of granite; the remainder of the stonework is Bramley Fall. There are two capstans at each end of the passage for hauling vessels through it.

The gates of the passage, Fig. 265, are of iron, with the exception of the heel-posts, meeting-posts, and sill-pieces, which are of greenheart timber. The gates are cellular, having two plate-iron skins separated by and riveted to horizontal and vertical ribs. The skins are formed with an outward curvature, but the sill-piece is straight, to fit the side. The sill of the lock is of granite, formed in two straight lines meeting at a point in the centre of the passage, and making together an angle of 126°. The heel-post works on a steel pivot 12 inches in diameter, let into the heel-post stone. Each gate is also supported by a roller, at a distance of 26 feet 8 inches from the heel-post, running on a cast-iron roller-path. The gates are secured at the top to the walls by strong iron anchors let into the masonry. After having been erected, the gates were so far filled with water as to counterbalance their ten-

Fig. 265.—Dock Gates, New South Dock.

dency to float. There are three sluices at the lower part of each gate, for adjusting the levels of the water in the dock and in the basin. The gates are opened and closed by means of chains passing over cast-iron rollers fixed in roller-boxes at the bottom of the chain-passages.

The passage from the dock leads to the basin, which is 600 feet long and 370 feet wide, and is surrounded by a quay wall, like that of the dock. It is, like the dock, puddled at the bottom. On the north side, the basin is connected with the south entrance of the Junction Dock, thereby forming a means of communication between the South Dock and the others of the West India Docks. The principal object of the basin is to serve as an immense lock during a rising tide. The water level is lowered to that of the river when the tide has risen sufficiently. The gates between the basin and the river are then opened, and the vessels are brought into the basin from the river, or *vice versâ*, until high water, when the gates are closed, and the vessels in the basin can be passed into the dock at leisure. By this system the water in the dock can be kept at a constant level.

The lock, Fig. 266, forming the east entrance from the Thames, near Blackwall, into the basin, is 300 feet long between the gates, and the width is 55 feet. The bottom of the lock between the gates consists of a segmental brick invert, 3 feet thick, laid on a concrete foundation, with springing stones of Bramley Fall. The gate floors and the aprons outside the gates consist of ashlar masonry of Bramley Fall, laid on foundations of concrete, the masonry being bedded in lias lime and grouted with Portland cement. The foundations under the sills are like those under the sills in the passage. The north wall of the old lock was left standing, and that of the new entrance was of solid brickwork 9 feet thick in front of it. The south wall is like the dock walls, except that outside the outer gates the walls are built of

solid brickwork to 16½ feet above the foundations to increase the strength at this place, where the pressure in front is constantly varying with the rise and fall of the tide, and where the walls are not supported, as they are between the gates, by an invert.

For the purpose of emptying and filling the lock, and also for lowering the level of the water in the basin, there are sluices in the walls on each side, at both pairs of gates, in addition to the sluices in the gates themselves. The inlets into each sluice-way are four in number, placed in the gate-recesses and on a level with the gate-floors. These open into the main sluice-ways, which are 5 feet wide and 7½ feet

Fig. 266.—Lock: New South Dock, West India Docks.

high and are over-arched; and they lead to the two outlets in the invert on each side. Beyond the entrance-gates, there are four outlets on each side, distributed along the walls, and so serving to clear away any mud which might be deposited on the apron. The main sluice-ways are lined with Staffordshire blue bricks; the sides and tops of the inlets and outlets are of Bramley Fall stone; the bottoms of the sluice-ways throughout are paved with this stone; the grooves for the shuttles, of which there are two in each sluice-way, are also lined with it. The gates, sills, hollow quoins, and chain-passages are the same as those for the passages already noticed.

Two timber jetties, on piles, one at each side of the entrance, are carried into the river for the convenience of incoming and outgoing vessels. Vessels lie alongside the southern jetty awaiting the opportunity to enter. Outgoing vessels are assisted by the northern jetty in bending their course down the river, which is at right angles to the entrance.

A railway swing-bridge crosses the passage, and a road swing-bridge crosses the lock.

Accommodation was provided for the gas-pipes and water-pipes supplying the Isle of Dogs. Two grooves were formed in the invert, and a shaft in each wall, to receive the pipes.

There is a capstan at each side of the entrance, outside the lock.

All the swing-bridges, gates, capstans, shuttles, and cranes are worked by hydraulic machinery supplied by Sir W. G. Armstrong & Co. In order to supply the increase of power required for working this machinery, besides the machinery already in operation, additional engines and boilers were erected at the engine-house at the West India Docks. A second accumulator was erected at the same place, and another at the south side of the new dock. The water was conveyed to the dock in 5-inch pipes.

The works were commenced in October, 1866, when a coffer-dam was constructed across the opening between the timber-pond and the canal. The water was then drawn off from the timber-pond, so that the foundations of the warehouses and the excavation for the new dock could be proceeded with before the water was excluded from the canal. A coffer-dam was begun at the same time across the canal, between the entrance to the Junction Dock and the site of the present passage, as the new east entrance lock did not form part of the original contract, and it was desirable to leave the portion of the canal, east of the Junction

Dock entrance, open for a time so as to have water accommodation for unloading materials for the works. In July, 1867, the foundations of the warehouses were completed, the tide was excluded from the canal, and the greater part of the water in it run off at low water, through sluices in a coffer-dam constructed across the west entrance lock; and, at the same time, the foundations for the south wall of the dock were laid. In August, the coffer-dam on the river-side of the old eastern entrance lock was commenced. It formed an arc of a circle of 150 feet radius, and was composed of a double row of piles separated by an interval of 5 feet, which was filled with clay, the sides of the coffer-dam being strengthened with laminated walings composed of 8-inch planks. The foundations of the walls rest partly on gravel and partly on stiff blue clay, mixed in some places with a good deal of sand. Along the site of a portion of the north wall of the dock, such a firm and thick bed of conglomerate was found that the concrete foundation was dispensed with, and the wall was laid on the conglomerate. The ground through which the dock excavations were carried consisted chiefly of a thick bed of gravel and sand in varying proportions, and averaging about 20 feet in thickness. The dock was opened on the 5th of March, 1870.

The cost of the work was as follows:—

Dock, basin, passage, east entrance lock, and warehouse foundations	£467,639
Warehouses	60,000
Blackwall and Millwall Extension Railway, portion within East and West India Dock Company's boundaries	24,500
Hydraulic machinery	19,000
Total cost	£571,139

The cost of the dock wall, as constructed, was only £12 2s. per lineal foot. The lowness of the cost was due, in a great measure, to the abundance of sand and gravel in the

excavations, with which concrete was largely made and employed, making the work less costly than brickwork would have been. The cost per cubic yard was 12s. 6d. The east entrance lock cost £95,000, inclusive of excavation, dredging, and coffer-dam. The excavation for the dock, passage, basin, and foundations of the warehouses amounted to 1,600,000 cubic yards, and cost £160,000. The gates cost £4,000 per pair; the jetties, £1,100 each.]

CHAPTER XI.

GRAVING DOCKS.

Graving docks (or dry docks) are docks constructed for the reception of vessels while undergoing repairs. They are usually made of such dimensions as to contain only one vessel at a time; their sides are formed in steps, so that the form of the dock is somewhat similar to that of the vessel which it is to contain, but sufficient space is left around it to enable the workmen to get at every part of the bottom of the vessel, and to afford sufficient light for the necessary repairs to be made. The entrance of the dock is closed with gates, precisely similar to those which we have described as belonging to canal locks, by which means, when the vessel has been floated into the dock and the gates closed, the water is pumped out of the dock, leaving it perfectly dry, the vessel being supported on timber struts and shores resting upon the steps already mentioned, as forming the sides of the dock. The accompanying illustrations are of a very fine graving dock, constructed by the American Government at their dockyard near New York. Fig. 267 is a longitudinal section, taken along the centre of the dock; Fig. 268 is a plan; Fig. 269 a front view of the entrance; Fig. 270 a transverse section through the centre of the dock; and Fig. 271 another transverse section through the recess for the lock gates. The dimensions of the dock are sufficient to contain the largest vessel in the American Navy, its length within the gates being

Fig. 967.

Fig. 968.
Graving Dock, New York.

820 feet, its breadth 98 feet, and the width of the lock gates 70 feet. The manner in which the vessels are sup-

Fig. 269.

Fig. 270.

Fig. 271.
Graving Dock, New York.

ported upon timber struts, when the water has been with-

drawn from the dock, is shown in Fig. 270, from which it will be seen that ready access is afforded to every part of the vessel. In order that the bottom of the dock may be at all times dry and free from water, it is formed with a slight inclination from A to B (Fig. 267), and a gutter is carried across the dock at the lower end, leading into a drain or culvert, C C, which passes entirely round the dock, as shown in Figs. 267 and 270, with a gradual fall towards D; and, the water being constantly pumped out of the culvert, it is impossible for any to accumulate at the bottom of the dock. Several flights of steps (E E E) are provided in different parts of the dock for the use of the workmen, by which they are enabled to reach any part of the vessel with great facility.

FLOATING DOCKS.

[The floating docks constructed by Messrs. G. and J. Rennie, at Cartagena and at Ferrol, are shown endwise in

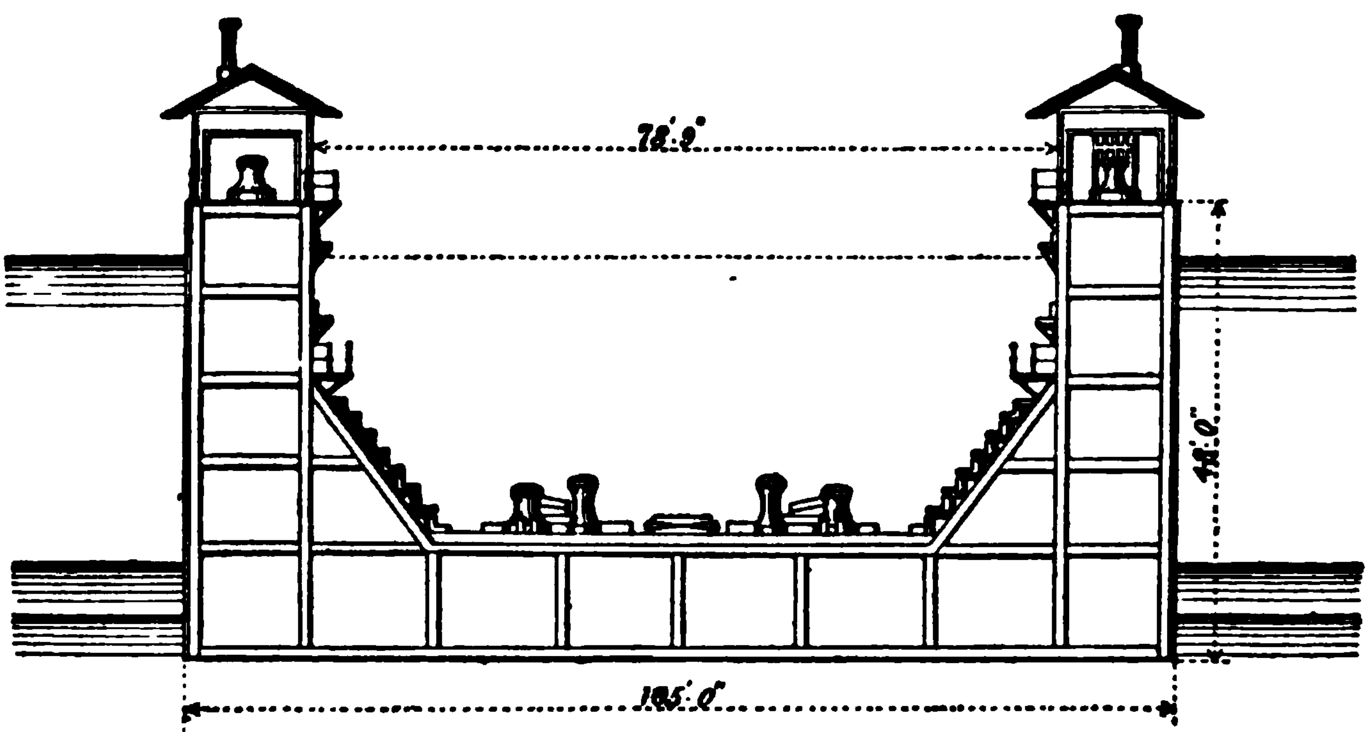


Fig. 272.—Floating Dock, Cartagena and Ferrol.

Fig. 272. They are necessarily constructed hollow at the bottom and the sides, as they depend for their buoyancy and lifting force on the air contained within them displacing so much water. The upper parts of the walls at the sides are

divided into compartments, forming permanent air-chambers, or floats, of a capacity sufficient to prevent the dock from sinking when the lower compartments are filled with water, and to maintain the decks of the side walls at a level of from 6 to 8 feet above the surrounding water. Mr. Rennie considers that these air-chambers are essential to the safety of an iron floating dock. The general dimensions of the two docks are as follows:—

	Cartagena.	Ferrol.
Length	324 feet.	350 feet.
Breadth	105 ,,	105 ,,
Height of Outside . . .	48 ,,	50 ,,

The basement, or lifting chamber, is, at Cartagena, 324 feet long, and at Ferrol 350 feet long; the breadth is 105 feet, and the depth is respectively $11\frac{1}{2}$ feet and $12\frac{1}{2}$ feet. It is made of $\frac{5}{8}$-inch boiler-plate, and is divided longitudinally into two equal parts by a plate-iron bulkhead $\frac{5}{8}$-inch thick. Each of these halves is divided by transverse bulkheads into ten equal compartments at Cartagena, and eleven at Ferrol, making respectively 20 and 22 water-tight compartments. Each of these chambers is subdivided longitudinally into two compartments by a partition which is perforated, so as to admit of a gradual flow of water transversely in the event of the dock suddenly listing over. The outer plating of the side walls is $\frac{9}{16}$-inch thick at the bottom, diminished gradually to $\frac{5}{8}$-inch at the top. For the work of pumping, a pair of horizontal steam-engines drive two pairs of lift-pumps immediately under the engine, to draw water from a common pipe communicating with all the chambers. On the ends of these pipes are fixed the sluices of the inlet for filling the chambers, and on the sides there are smaller sluices and pipes in communication with each chamber. By opening all the sluices, the chambers are filled; and on shutting the inlet sluice with the engines at work, one or many chambers may be discharged. The whole power of the engine may be directed

to pumping out any one compartment, when it is found desirable to do so in order to balance or level the dock. There are distinct pumping arrangements for each side of the dock.

The engines and pumps are of the following dimensions. The engines were designed to make 60 turns per minute; the pumps, by means of gearing, half the number:—

	Cartagena.	Ferrol.
Diameter of cylinders .	14 inches.	18 inches.
Stroke of pistons . .	18 „	19 „
Diameter of pumps . .	20 „	24 „
Stroke of pumps . .	33 „	36 „

The spindles, with the columns and hand-wheels of all the sluices, are carried to the upper deck within the engine-house, and they can all be manipulated within a space of 15 feet square.

The air-pipes are 6-inch cast-iron socket-pipes, carried up from each basement and middle chamber to the upper deck, for the exit and the entry of air during the filling and discharging of water into or from the chambers.

The floor of the dock is covered with 3-inch teak planks, resting on cross beams of teak 2 feet square, placed 16 feet apart, one on each transverse bulkhead and an intermediate bearer. The keel-blocks are of teak, with cast-iron wedge pieces.

The total weight of the floating dock is 400 tons, and the draught when empty is 4 feet 7 inches.

The basin, or dock receiver, in which the floating dock receives the ship, is 345 feet long; it is curved at the landward end, from which three lines of horizontal ways or slips radiate. Each line is 725 feet long and 45 feet wide, constructed with two altars, 5 feet 9 inches wide and 10 inches high, thus making the floor of the ways or slips 19½ inches below the surface of the ground. Each is laid with four lines of timber ways, about 10 feet apart, with keel blocks to

correspond with those in the floating dock; and is intended to receive vessels, after they have been raised by the floating dock, from the dock into the slip by means of hydraulic power. It is estimated that six vessels may be building or be under repair at the same time, besides one on the floating dock.

Vessels of from 4,600 tons to 5,600 tons can be floated by this dock. The largest vessel which has been docked [1871] at Cartagena is the *Numancia*, iron-clad, 316 feet long. The draught of the dock with this load amounted to 11¼ feet, with a depth of 7½ feet of water in the basement, and 7 feet 2 inches in the middle chambers, amounting to a weight of 800 tons. The distributed weights were, then—dock, 4,400 tons; water, 800 tons; ship, 5,600 tons; total, 10,800 tons.

The dock when hauled out, empty, sunk to a depth of 37 feet in 1 hour 25 minutes after the sluices were opened. It was lifted, with a ship aboard, in 3 hours.

The floating dock cost between £150,000 and £160,000, or about £35 per ton of its gross weight.*

The ingenious arrangement of Mr. Edwin Clark, by which the vessel is raised entirely out of the water by hydraulic pressure, and subsequently, with its pontoon, floated into one of a series of shallow docks above quay level radiating from the lift, is admirably adapted for a tideless sea. The Thames Graving Dock was constructed on this system. The lift is a direct mechanical appliance for raising the vessel by means of vertical hydraulic presses. It consists of two rows of cast-iron columns, 5 feet in diameter at the base, 4 feet above the ground level, and sunk into the ground about 12 feet. The two rows are 60 feet apart, and the columns of each row are 20 feet apart. There are 16 columns in each row, making

* See Mr. G. B. Rennie's paper on the "Floating Docks for Cartagena and Ferrol," in the *Proceedings of the Institution of Civil Engineers*, vol. xxxi. p. 295.

a length of 310 feet to the dock. The rows are placed one on each side of the excavated lift-pit in 27 feet of water. A 10-inch hydraulic press is enclosed in each column, with a length of stroke of 25 feet. The presses are connected in pairs, one on each side, by chains to cross girders, 16 pairs of which lie at the bottom of the pit, when the presses are lowered. They form a large platform or gridiron, which can be raised or lowered as required, with a ship in it.]

PART III.

HYDRAULIC ENGINEERING.

CHAPTER I.

SUPPLY OF WATER TO TOWNS.

The questions which affect the choice of the source and means of supply of water to towns are those connected with the qualities of the water itself, in the first instance; and, in the second, the relative conditions of the difference of level, and the distance between the source resorted to and the place in which the distribution is to be effected. All waters, as is well known, are not equally adapted to domestic use; and those which are so adapted are rarely found in the precise localities where they are to be used; so that in almost all cases it is necessary either to bring the supply from a distance, or to raise them above their natural level.

Notwithstanding all that has been said in the controversy respecting hard and soft waters, there is still very great uncertainty as to the precise qualities required in those to be distributed in towns; and the public cannot be too frequently advised to hesitate before it adopts implicitly the opinions of men who, though neither engineers nor physiologists, have lately assumed to dictate upon the subject. Unquestionably excessive hardness is an objection to a source of supply; but some of the chemical combinations which give rise to

this characteristic, if they only act within certain limits, are stated by the most eminent authorities to render the slightly hard waters more adapted for human consumption than any others. Soft waters, again, are unquestionably more pleasant and agreeable for domestic use than hard waters; but their very softness may be owing to the presence of ingredients able to affect, slowly, but surely, the physical organization of those constantly using them. Habit modifies the action of particular waters upon the human frame; and it is notorious that those accustomed to any one (whether soft, as flowing from the primary rocks, or hard, as affected by the carbonates or the sulphates of lime) are always seriously affected when they begin to use what would be universally considered a better water. Dogmatical assertions are as dangerous in this case as in all others; and, at least until competent authorities shall have decided what really constitutes the perfection of a water supply, questions of economical expediency must ultimately decide the course to be adopted in the majority of cases.

In the present state of uncertainty attached to this subject it may suffice to adopt the conclusions laid down by Thénard, and to pronounce those waters to be fit for domestic use which are fresh, limpid, and free from smell—able to boil vegetables without affecting their colours, and to dissolve soap without leaving curds. They should be very slightly affected by the nitrate of baryta, which will indicate the presence of the sulphates in combination; by the nitrate of silver, indicating the presence of the chloride of sodium; by the oxalate of ammonia, indicating the presence of the salts of lime; by the ferro-prussiate of potash, indicating the presence of salts of iron; or by the other chemical tests usually employed. The residuum, after evaporation, should be very small. A certain proportion of carbonic acid gas is considered to improve the digestive properties of water for drinking purposes; and nearly all physiologists, from the

time of Hippocrates to the present day, assert that, in small quantities, the chloride of sodium and the bicarbonate of lime are also essential.

The temperature of water is of nearly equal hygienic importance with its chemical nature, and it should be as constant as possible; that is to say, compared with the atmosphere, it should be warm in winter, and cold in summer. Aeration is an important condition, for the oxygen thus communicated forms, in fact, an essential element of the salubrity of water. Vegetable and animal matters, either in suspension or solution, must be removed; not only because they are disagreeable in themselves, but also because they absorb the oxygen in suspension in the water, and cause the latter rapidly to putrify. The presence of this class of impurities may be detected by chlorine solutions, or by an infusion of gallic acid.

After all, the most efficient method of ascertaining the real qualities of a water supply is, to observe the effects it produces upon the organized life resorting to it, especially upon the human beings using it. Organized life is, in fact, a far more delicate test than any chemical agents can ever be; and it is eventually affected by impurities too minute to be ascertained by the grosser appliances of science. Such waters, therefore, as are habitually used by vigorous, powerful, and healthy populations, can never be pronounced to be unfit for domestic consumption, whether they be hard or soft, or whether they contain salts of lime, or salts of soda, or potash.

Rain water, collected in the open country or at sea, a short time after the commencement of a shower (for the first drops that fall carry down the impurities in suspension in the lower strata of the atmosphere), is the purest that can be obtained. In storms it sometimes contains nitric acid; on the sea-coast it is often brackish; at all times it is aerated, but flat, and insipid to the taste, and apt to cause colics,

probably from the absence of carbonic acid gas. For industrial operations it is generally considered to be the most advantageous.

Snow water is without air, and usually deposits a small quantity of dust on being melted; ice water is bright and pure, but difficult of digestion. The loathsome disease, the *goître*, is usually attributed to the use of snow water; but the healthy state of the crews of Captain Parry's ships during their long arctic voyages, when they had no other resource than the dissolved ice and snow, would appear to show that, if proper precautions be taken, they may be resorted to without inconvenience. Indeed, as several of the sources of soft water from the earlier secondary formations produce glandular swellings analogous to the *goître*, it would be reasonable to infer that the latter must be owing to the matters contained in the snow, rather than to any qualities inherent in the waters derived from it.

Spring waters depend for their qualities upon the nature of the strata through which they pass. They are fed by the rain-fall soaking partially into the ground at a higher elevation, and finding its way to the surface at such points as offer less resistance to its escape than it meets in any other direction. As pure water, such as falls from the clouds, has a remarkable affinity for many of the earthly bases, and for the gases with which the latter combine, it must be evident that the springs will become impregnated with both, in proportion to the time they are exposed to their influence. Rivers, again, are formed by the confluence of springs and small streams fed by the drainage from the watershed; consequently, near their sources their waters must participate in the respective properties of the latter. In their course, however, they may acquire a degree of purity far greater than exists in the several affluents; especially if they run over a rocky or a sandy bed, and do not receive any organized matters draining from the lands they traverse. Much of the

gas taken up by the springs in their underground course may be given off in this manner, and even many substances in a state of chemical combination may become separated. It is not, therefore, always true that the nearer the source the purer is the supply; but, on the contrary, so much do waters often gain by exposure to the atmosphere, that many physiologists are of opinion that river waters are preferable to those obtained from springs.

The extent to which waters are improved by exposure to the atmosphere must naturally depend upon the nature of the impurities they may contain. Thus the carbonic, and sometimes the sulphuric acid gases, are parted with easily, and the earthy carbonates deposed; but the sulphates of lime and the chlorides of calcium and magnesium are retained much longer. Often the distinguishing elements of two streams may be traced for a considerable distance below the point of confluence; and, again, it may frequently happen that the impurities contained in either of them may facilitate the deposition of those contained in the other. The greatest practical inconvenience attending the use of waters taken directly from their source appears, however, to lie in this—that they are, under such circumstances, certain to depose their earthy salts in the conduits employed in their distribution. Such waters as contain the bicarbonate of lime, or the hydrous oxide of iron, are especially exposed to this objection.

Collection of Water from the Surface of the Ground.

The small streams collected from the watershed of a country must be affected by the considerations above described; that is to say, their qualities must depend upon the strata over which they flow, upon the organic matter carried into them, and their exposure to the atmosphere. It must be borne in mind, in all these discussions, that it is

now generally admitted that, of the total rain-fall supplying the fresh water of a large tract of country, one-third flows off in the shape of rivers above ground, one-third is employed by the vegetation or again evaporated, whilst the last third penetrates the ground to supply deep-seated springs and wells; or, at least, that these proportions hold for the majority of cases. It is the former quantity, therefore, that must be calculated upon, in all cases where it is proposed to obtain a supply from the watershed of any particular district; at least, unless it be possible also to secure any deep-seated springs.

Now, it must be evident, that as the streams from the watershed of a district are supplied by the rain flowing immediately off the land, they must vary considerably in volume; and that in winter, or the rainy season, they will be full, whilst in summer they will be comparatively dry. The variations in volume will depend upon the greater or less equality of distribution of the rain-fall, upon the configuration of the country with respect to the outlines of hill and dale, and upon the capacity of the superficial strata to absorb and retain water during wet weather and to part with it during droughts—in fact, upon their capacity to store water, and thereby equalise the flow. All these combined causes have been observed to produce very great irregularities; and it becomes necessary, when a constant equable supply is to be obtained from a given watershed, to construct reservoirs so as to store the excess of one period against the penury of another. The dimensions of the reservoirs must depend upon the distribution of the rain-fall, and it may be laid down as a rule, that they should be calculated more with reference to the maximum demand and the minimum supply than to the average of either. A capacity of storage equal to about six months' consumption, in addition to the quantity which is likely to be evaporated, appears to be the least which should be admitted when it is

proposed to supply any agglomerated population in this manner.

Waters thus stored are much exposed to deteriorate in quality. They develop with singular rapidity both animal and vegetable life, and the decomposition of the latter, when in a state of decay, communicates elements of future stagnant waters; and these, if exposed to the atmosphere, must also be exposed to the variations of temperature of the latter.

The following rules should be observed, wherever local circumstances will allow, in the construction of reservoirs: that the capacity be obtained by increasing the depth rather than the surface; that the sides be as nearly as possible vertical; and that they be covered, so as to protect them from atmospheric influences, amongst which may, perhaps, be included the sun's light, for it appears to be the most efficient cause in promoting vegetation. The expense attending the execution of these works is so enormous that there can be but very few cases in which they ought to be undertaken; and, indeed, in all cases where covered reservoirs are required, very careful and elaborate comparative estimates of the cost of all other sources of supply should be made.

It may be interesting to state that the cost of some large canal reservoirs has been about £450 per million gallons of water stored. No town reservoirs appear to have been constructed at a less cost than about £600 per million gallons; whilst the Croydon Reservoir, the only covered one yet constructed, cost rather more than £4,000 per million gallons, including the price of the land.

A well-executed system of catch-water, or of thorough drainage, would unquestionably increase, to a remarkable extent, the quantity of water derivable from a given area, but it will in nowise diminish the necessity for constructing the storage reservoirs, but rather, on the contrary, augment it; for, necessarily, the storage capacity of the ground itself

is diminished in the direct proportion of the perfection of the drainage. The formation of a system of thorough drainage is also a very expensive operation; and, when combined with the necessity for reservoirs, it must lead to so great an outlay, that it may safely be asserted that the system of collecting water from what it has lately become the fashion to call "gathering grounds" should never be resorted to, if any other can reasonably be adopted. However, it has been calculated that if the situation of the proposed gathering grounds offer steep declivities with narrow gorges, it is possible to obtain from it two-thirds of the total rain-fall—although hitherto no instance can be cited where such favourable results have been long obtained, for the efficiency of the drains becomes rapidly deteriorated.

Use of Springs.

Where springs, fed by the infiltration of rain-waters falling upon a large area, occur in considerable abundance, and of the requisite quality, they may be considered to offer the most desirable sources of supply in all highly cultivated districts, because it is hardly possible to exclude drainage waters from flowing into superficial watercourses. The precautions, before alluded to, as being necessary to secure the deposition of the matters in solution which would otherwise choke the pipes, must be taken. It is also important to ascertain what is called the yield of the springs under all the varying meteorological conditions of the locality. Gaugings of any watercourse during two or three months are of very little use in cases of this kind, because the springs being supplied by the rain-fall it follows that they must vary with the variations of the weather. Unless observations, then, be extended over the whole cycle of the climate (in England, of about seventeen years' duration), the indications to be derived from gaugings over short periods are very likely to

mislead in calculations with respect either to the average or the minimum flow. The period of the year when gaugings are taken will also materially affect the correctness of the observations, because not only does the rain-fall vary with it, but also the springs are observed to be affected at an interval of from one to five months, according to the nature of the strata. As the greatest quantity of rain falls during the later autumnal and the winter months, it appears that, if it be necessary to confine the observations within very short periods, the most advisable course is to make them during the months of September and October; for, generally speaking, the springs are at the lowest about that period. Even then the indications are very likely to be fallacious, and, unless observations be carried over the whole cycle, the yield may occasionally fall far short of that calculated upon.

Artesian Wells are, in fact, nothing more than excavations through the overlying impermeable stratum supposed above to exist upon a permeable water-bearing stratum, underlain again by an impermeable one. They form, as it were, artificial outlets for the waters contained in the lower parts of the basin, and the water-line in them will depend upon the hydrostatical pressure existing upon the same lower parts. This, as said before, will be influenced by the level of any natural overflow which may exist around the edge of the retaining stratum. The conditions of success in an artesian well depend upon the perfection of the basin formed by the water-bearing stratum, so far as the mere retention of the water is concerned, and upon the level of the streams flowing from the water-bearing stratum, so far as the height of the water-line is concerned.

The quantity of water to be obtained from an artesian well must, therefore, be regulated by the area of outcrop of the water-bearing stratum; and that it is far from being unlimited is proved by what has occurred at London, Tours, and Milan.

The various physical conditions necessary to insure the success of artesian borings also introduce great uncertainty in their results. Thus the well of Grenelle yielded a copious supply at the depth of 1,802 feet; in the valley of the Loire several successful works have been executed, with an average depth of 500 feet; whilst one other, in the same district, of 628 feet in depth, and a second, 454 feet deep, were abandoned, after the usual retaining basin of the district had been traversed, on account of the absence of supply. At Calais, an artesian boring, 1,150 feet deep, was made unsuccessfully through the chalk and subcretaceous formations into the carboniferous series. At Chichester, at a depth of 1,054 feet from the surface, no water was obtained from the upper green sand; and at Southampton the boring was discontinued.

The only infallible source for a water supply is to be found in rivers, although unquestionably there are often serious objections to their use. Thus it almost always happens that they receive the drainage waters from cultivated lands and inhabited districts; and the daily-spreading habit of making their beds serve as the outfall for sewage tends seriously to increase this evil. When rivers are resorted to, it becomes, therefore, necessary to place the supply-conduits at points beyond the reach of such sources of impurity, and, in almost all cases, to form settling reservoirs and filter beds, so as to remove the extraneous matters present in their waters.

The quantity of water to be distributed in a town is usually assumed to be about at the rate of 20 gallons per head per day, calculated upon the whole population. This quantity would include all that is generally required for municipal and trade purposes, unless when the latter are of the character of cotton printing, dyeing, or scouring cloths, &c. In London the supply is rather beyond that quoted above; in Paris, also, it is above 20 gallons; but,

where accurate observations have been made, it appears that really the average daily consumption for every human being is about 7 or 8 gallons, and that in summer they require from 20 to 30 per cent. more than in winter. Municipal requirements vary, of course, according to the habits of the country, but they rarely exceed 10 per cent. of the total consumption; and ordinary trade purposes, together with the inevitable waste, make up the remaining quantity short of the 20 gallons per individual per day usually assumed to be required.

The parties considered to come under the designation of large consumers, and as such giving rise to an extraordinary demand for water, are manufacturers, tanners, fell-mongers, hair-washers, glue-makers, curriers, dyers, hatters, brewers, distillers, inns, bath-houses, and steam engines. If many such exist, it will be necessary to provide especially for them; which, of course, would place the rate of payment upon a different footing from that to be supplied to the public in general.

[In proceeding to the consideration of means for effecting a water supply, the extent of the drainage area is ascertained, and the amount of rainfall is arrived at, by taking the average of the observations for a number of years in the neighbourhood. Then, the quantity of rainfall that can be collected into any reservoir which it is practicable to make in the district, is to be considered. Assuming the case of a district where a reservoir can be constructed, capable of storing nearly all the water that can be collected, the capacity of the reservoir should be proportioned to the population to be supplied. Its area is frequently about one-twentieth part of that of the water-shed. It is almost impossible by means of reservoirs to govern the vast floods that occasionally occur; and as those will happen, sooner of later, when the reservoirs are already filled, as a matter of

course, an uncertain quantity of water will flow over the waste-weirs and be lost, not only to the waterworks, but it will be lost as regards any useful application whatever, by the mill-owners or others situated on the stream. This unavoidable loss is often estimated at 10 per cent. of the residue, after deduction made of the amount lost by evaporation from the surfaces of the land and the water. The loss by evaporation, it has been found by experience, depends upon the nature of the surface, the inclination of the land, the amount of vegetation with which it is covered, and other circumstances. As a general rule, according to Mr. Hawksley, the evaporation, upon an average of years, varies from 12 inches on a precipitate rocky country, to 18 inches on land which is of a flatter character, or which may be of a more absorbent nature. If the evaporation, which can generally be nearly accurately estimated by a survey of the drainage area, be assessed at 15 inches, which is about the average in England, then there can be collected, on an average of years, about 30 inches out of a rainfall of 48 inches.

The next thing to be determined is the size of the reservoir to receive and to make this quantity of water available. It is regulated by the amount of the rainfall, which is different in different parts of the country. Where there is a large quantity of rain, a smaller number of days' storage will suffice than where the fall is more restricted. For instance, on the hills in Lancashire, where the rainfall is from 45 to 48 inches per annum, the reservoir need not be capable of storing more than 140 days' supply; in Cumberland, where the rainfall amounts to 60 inches per annum, 120 days' supply would suffice; but in the eastern parts of England, where the rainfall does not exceed 22 inches per annum, it is necessary to make a reservoir capable of containing 200 days' supply. With these data, the amount of capacity of reservoir required to utilise the water can be

decided with considerable precision. As to the amount to be supplied, if the waterworks are well managed, and care is taken to prevent waste, 16 gallons per head per day is sufficient for usual sanitary, manufacturing, and domestic purposes; or, where the manufactures are in excess, then from 20 to 25 gallons should be allowed. But where the internal fittings are not properly looked after, that amount of water may be increased to from 30 to 60 gallons per head per day.]

CHAPTER II.

RESERVOIRS FOR WATER SUPPLY.

The site to be chosen for the reservoirs must be, so far as economy of construction only is concerned, as near as possible to the source of supply, if the latter be situated at a low level; or to the commencement of the distribution, if the waters be led from a great distance. It is advisable, also, wherever it can be conveniently effected, that all such reservoirs be placed out of the reach of the impurities of the atmosphere of all large centres of population; and that they be protected from dust and soot, as well as great changes of temperature. These considerations are often of so great importance, that they may cause it to be preferable to augment the engine-power, and construct the reservoir at a lower level, even if they do not lead to the abandonment of the latter altogether.

In addition to the remarks already made, with respect to the construction of reservoirs, in this and the preceding chapters, it is necessary to state, that whenever such works are to be executed for a town supply, they must be formed of such materials as are not likely to affect the qualities of the waters they are intended to receive. In all cases of this description, to a certain extent the waters must be stagnant, and they are then likely to absorb any soluble salts contained either in the earth or in the masonry of the wall. It appears, therefore, to be very doubtful whether water intended for town distribution should be stored in reservoirs which are puddle-lined or pitched with calcareous stones. Silicious

sandstones, hard-burnt bricks, or the argillo-calcareous stones, bedded, where necessary, in powerfully hydraulic limes, or in cements, or iron, protected from the immediate chemical action of the water, are unquestionably the most advisable materials to be used in forming the faces immediately in contact with the water. The positions of the inlet and outlet pipes should be arranged in such a manner as to insure a constant flow through the body of water in the reservoir; and precautions should be taken to keep back any impurities which might be introduced, by either forming depositing-wells under the inlet-pipes, or by placing gratings or filters over the heads of the outlets.

The other accessories to reservoirs intended to hold waters for town distribution are simply—1, the valve-pit, placed at a small distance from the outlet through which both the pipes are made to pass if possible: it is formed for the purpose of working and examining the respective valves by means of which the water is admitted to or excluded from the reservoir or the pipes, as the case may be; 2, the overflow-pipe, waste-weir, or other provision for regulating the height of the water; 3, the scouring or cleansing-pits, with a discharge-pipe placed at such a point as to allow the whole of the water to be drawn off if requisite; and, 4, means of access to the bottom of the reservoir. It is desirable, and practically it is almost always so arranged, that the outlet-pipe be so placed, that a certain depth of water should always be retained, excepting when the cleansing-pipe is opened. The object proposed by this arrangement is to allow a more effectual deposition of the mechanical impurities of the water.

[In constructing the dams of reservoirs, the system usually followed is to excavate the ground to a depth sufficient for removing from its area all material of a boggy, peaty, slippery, or compressible nature, as well as any silt or earth, which, when acted on by water, is likely to run into a quick-

sand. In the centre of the dam, a puddle wall, or core, is formed of tough impervious clay, let into a trench which is dug out to receive it, and carried up to the level of the top of the embankment. The thickness of the core is dependent to some extent on the total height of the embankment. The width at the top is made of various widths of from 5 feet to 10 feet. It is made with a batter on both sides which is generally such that the thickness at the base is about one-third of the depth of the water to be sustained. A good rule is to make the width at the top 10 feet, and to give the sides a batter of 1 inch per foot. For an embankment rising 6 feet above the water, to sustain the pressure of 66 feet of

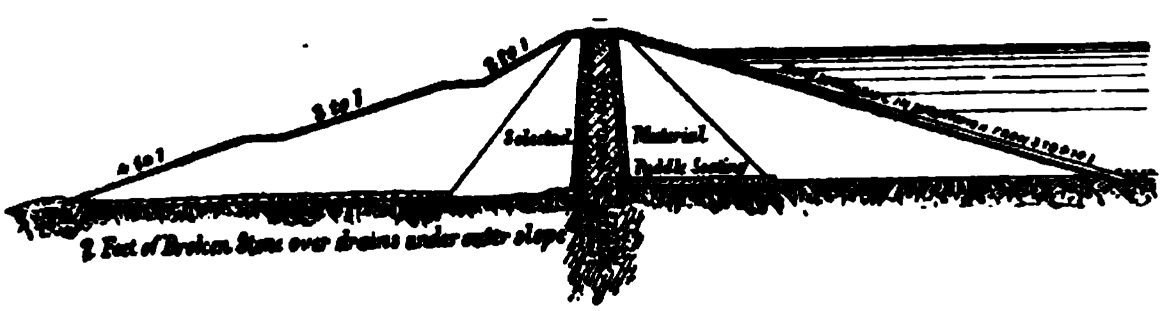


Fig. 273.—Dam for Reservoir : Normal Section.

water, the thickness at the bottom would, by this rule, amount to 22 feet. The puddle wall is immediately supported at each side by material selected for its retentive character, and the remainder or outer portion of the bank is made up of the harder or less impervious earths. The inner slope is covered with stone pitching, to prevent injury to the embankment by the wash of water. The normal section of such a dam is illustrated in Fig. 273.

Dams constructed of masonry are better and safer than earthen dams, provided that they are built upon a continuous foundation of rock. They are adopted in countries where the hard, primitive, basaltic rocks prevail, and where special provision against the percolation of water is not demanded.

The outlet for water from the reservoir it was customary, formerly, to carry through and under the earthwork of the embankment. On this system, there is a constant risk of the

leakage of water along the surface of the pipe or culvert itself, by the unequal pressure of the embankment, and by the unequal settlement to which it is liable at the place where it traverses the puddle trench. Naked cast-iron pipes, thus exposed, are peculiarly liable to fracture if they be only surrounded by puddle; and, if they be commanded by a valve at the outer end, being thus constantly and necessarily full of water, the risk of damage is evidently greater than when the shut-off valve is placed at the inner end of the pipe. But, even where the pipe is carried through a culvert of masonry or of brick, the culvert itself is exposed to the strain of unequal settlement at the centre where it crosses the puddle trench.

In the opinion of Mr. A. R. Binnie and other engineers, the best method of providing an outlet, and at the same time obviating the risks of failure above noticed, is to design the embankment so that it can itself settle, and may compress the ground on which it stands, without either doing injury to itself or to any other part of the work, and to design the outlet in such a manner that it shall be perfectly under control, and that any leakage from it will not injure the embankment. For the attainment of these objects, tunnel-outlets have been constructed, formed of masonry or of concrete and iron, placed in a drift or adit, which has been regularly mined. The course of the tunnel is carried either round and clear of the end of the embankment; or, if passing under any part of it, it is situated at a considerable depth in the solid rock below the base, not only of the embankment but also of the puddle trench. The flow of the water through the tunnel is generally regulated by a valve-tower in the reservoir at the inner end of the tunnel, or by valves built in a well situated about midway in the length of the tunnel, and quite clear of the embankment. The first is the better system, as it provides for shutting off the water from the whole length of the tunnel, when the tunnel is required to be clear for inspection or repair.]

Should the only available source of supply be of a quality to require filtration, it will be found advisable to perform that operation at the very latest possible period before the distribution takes place. However perfectly filtration be performed, whether by chemical or merely by mechanical agents, if the water be stored for any length of time afterwards it will infallibly develop or take up impurities. The course adopted by some of the London companies—filtering the water, and then raising it into reservoirs, where it is exposed during several days to the action of the atmosphere—is radically incorrect. In such and in all similar cases the water should be allowed to settle at the lower level, and the filtration should take place after it leaves the upper reservoirs. Precautions, however, must be taken to insure that all impurities likely to choke the pumping-mains be removed before the water enters them.

Wherever filters are used, it is customary to construct before them settling reservoirs, in which the waters may deposit the grosser impurities they may contain; and it appears advisable that these should be of sufficient capacity to allow the water to settle during at least three days. From thence it must be led upon the filter, without velocity or current able to act upon the materials of which this may be composed. The filters themselves may be either chemical or mechanical; that is to say, they may either alter the qualities of the water or they may merely act by removing impurities in suspension. To effect the former is necessarily a costly and difficult operation, and, indeed, so much so, that it may fairly be considered unattainable with the whole quantity required for a town supply. And if the water be immediately taken from a good mechanical filter, with as great rapidity as this can yield it, the quality will, in almost every case, satisfy not only the public demand, but also the real exigencies of the case.

CHAPTER III.

FILTRATION FOR WATER SUPPLY.

[For large towns where filtration is necessary, by the system generally adopted the water is filtered downwards, passing by its gravitation through layers of sand and gravel. For this purpose shallow reservoirs are constructed, of which a common form is shown in section, Fig. 274, usually lined with brick, with inclined sides, and a floor nearly flat. Drains are formed on the floor and are covered with

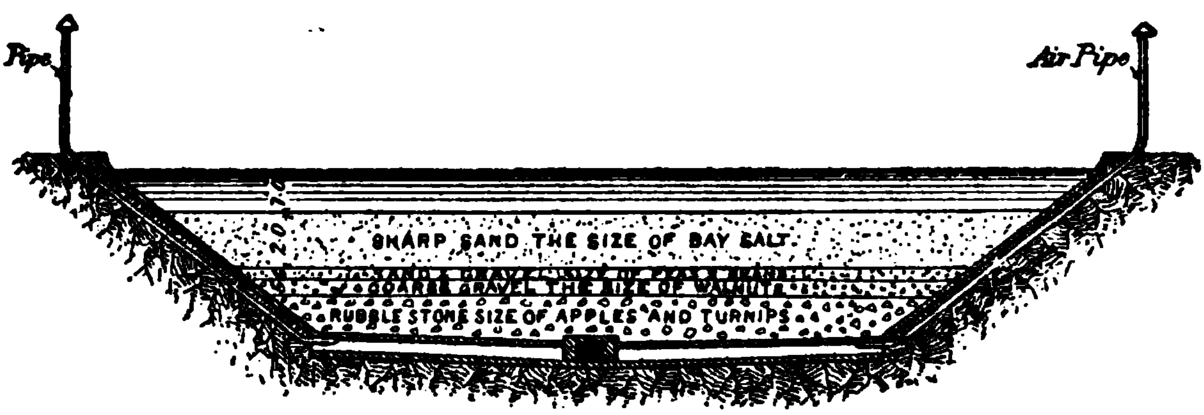


Fig. 274.—Filtration Reservoir.

filtering material to a depth of about 4 feet. At the top there is a layer of 2 feet of coarse granular silicious sand, insoluble in water, of the size of bay salt; next there is a layer, 6 inches deep, of coarse sand or of fine gravel about the size of peas and beans; then a 6-inch layer of coarser gravel of the size of walnuts; and at the bottom a layer, 12 inches thick, of rubble stones about the size of potatoes and turnips, surrounding and covering the drains. This filter is cleansed from time to time by removing a few inches of the upper stratum of sand, which after having been washed can

be replaced on the filter. In forming a filter-bed care should be taken that the sand is not fine, otherwise the filter speedily becomes choked. Coarse granular sand attracts the particles of matter suspended in the water just as well as the finest sand.

The pipes which lead from the drains under the filter to the pure-water tank should, according to Mr. Binnie, be so arranged that the head of water on the surface of the sand should not exceed 12 inches, for the success of the process of filtration is contingent on the slowness with which the water passes through the filtering bed. Mr. Bateman has found in practice that such a filter can properly purify water when the rate of progress does not exceed 6 inches vertically per hour, equivalent to a discharge of 75 gallons per superficial foot per day of twenty-four hours. By this measure 1,500 square yards of filtering area should be provided for every 1,000,000 gallons required per day, and an allowance in addition as provision for setting off the filter from time to time for cleaning and repair.

Mr. Hawksley is of opinion that sand filters operate not only mechanically but also chemically, and that the chemical changes depend very much upon the particular state of the organic matter in the water, and on the admission of the free oxygen of the atmosphere. The sand filter clears the water mechanically by the agency of the well-known principle of attraction and aggregation. When the water descends slowly through the filter, the minute particles of matter suspended in it are attracted by the facets of the sand, and they adhere there. Thus the water becomes clear, and the operation is carried on so rapidly that scarcely in any filter does the water remain foul for more than a few inches below the surface of the filter bed. A thin bed of sand is therefore considered to be just as effective as a bed of 2 feet or 3 feet in thickness. The sand filter also acts chemically, for it actually destroys organic matter, and it deprives water in

which lime is held in solution by the excess of carbonic acid of part of the chalk, thereby softening the water. Any substance which attracts oxygen to its surface and renders it to organic matter, destroys the organic matter, and renders the water pure. This action is, to some extent, set up in the sand filter.

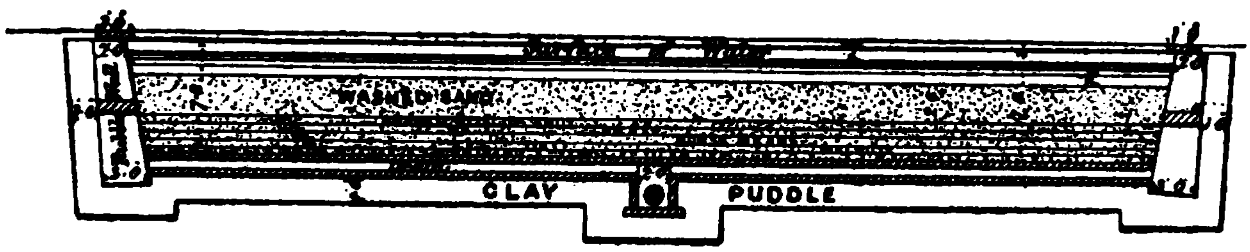


Fig. 275.

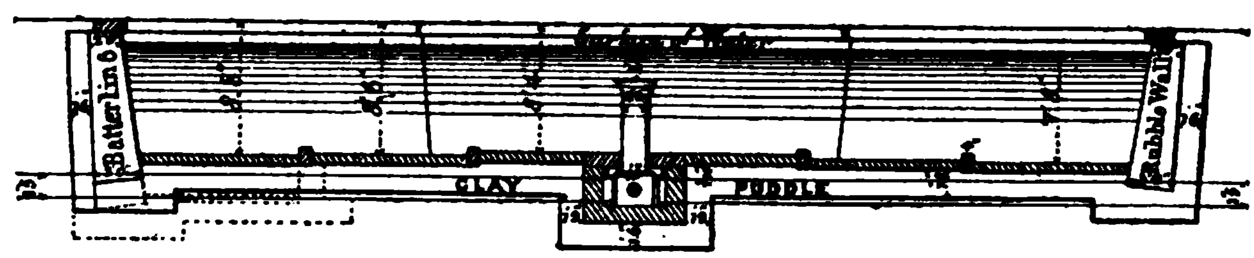


Fig. 276.
Filter Beds and Pure-water Tank, Leicester Waterworks.

The arrangement of filter-beds and the pure-water tank of the Leicester Waterworks, designed by Mr. Hawksley, are illustrated by Figs. 275 and 276. Before filtration, the water holds in suspension a perceptible quantity of muddy matter, and is also sensibly brown from a little peat and other organic matter found on the surface of the watershed from which the water is derived. After filtration, the water is completely freed from matters in suspension, and is generally deprived of colouring matter, though in the worst seasons there is sometimes a slight discolouration. The water is brought from a large storage reservoir, about 40 acres in extent, into the filter-beds, four in number, in separate channels, so that either filter may be cut off and cleaned without disturbing the action of the others. The filter-beds are each 99 feet long, 66 feet wide, and 8 feet 8 inches deep. They hold sand to a depth of 2½ feet, upon 2½ feet of gravel, varying in size from that of beans to that of eggs. The water then

passes down through the sand and the successive strata of gravel, into the drains, and from these, by means of pipes, into the central pure-water tank. This tank is 8 feet 8 inches deep, and holds 7 feet 8 inches of water. To prevent the pressure exceeding a certain limited extent, the pipes deliver the pure water into a pipe which stands up in the pure-water tank within 2 feet of the surface. By this contrivance there can never be more than 2 feet of pressure upon the sand, and the water is obliged to go through at a slow rate, amounting to about 600 or 700 gallons per day per square yard of surface, which is equivalent to 66 or 77 gallons per square foot per day. The water thus filtered is not only free from suspended matter but is to some extent operated upon chemically, so as sensibly to diminish the proportion of organic matter.

The water supplied to the inhabitants of Berlin is derived from the river Spree. Previous to its being received into the tank or reservoir for distribution, the water is passed through filters and is freed from impurities; but, after remaining in the open tank for a day or two, the whole surface of the water got covered with confervæ and vegetation of various kinds. The water also lost its transparency, and became thickly turbid. The engineer thereupon recommended that the pure-water tanks should be covered over. They were covered, and immediately the unpleasant symptoms disappeared, the water remaining in its pure condition. Mr. J. F. Bateman reports a similar occurrence at the Warrington Waterworks, where the water was collected from gentle slopes of cultivated land of the New Red Sandstone formation into a reservoir, after which it was rendered perfectly pellucid by being filtered through sand, and was then passed into a tank not more than 6 feet deep. Although that water was filtered as well as it could be, it very soon became filled with animalcules, for the separation of which Mr. Bateman introduced a copper wire-gauze strainer of eighty strands to the inch. But

it was so difficult to cleanse the gauze from the animalcules, that he ultimately resorted to the covering over of the reservoir. This was effected without emptying the reservoir by erecting vertical pillars supporting cross beams, on which large flags were placed. As the work progressed the confervæ beneath the covered parts gradually fell to the bottom, and by the time it was finished they had entirely disappeared, whilst the water has been delivered in a pure state ever since.

The growth of confervæ is peculiar to shallow waters. In tanks of from 15 feet to 20 feet deep they are not generated. The service reservoirs of the Manchester Waterworks, which are lined with bricks throughout, are 18 feet deep at one end and 20 feet deep at the other end. They are kept constantly filled to within a foot or two of the top, and no vegetation appears upon them.]

CHAPTER IV.

MODE OF DISTRIBUTION OF WATER SUPPLY.

When the source of supply shall have been determined upon, it becomes necessary to consider the system of distribution to be adopted. Formerly, in almost all English towns, this was effected by what is called the intermittent system, in which the water was supplied from the mains during a greater or less number of days in the week, and stored in cisterns for domestic use in the intervals. This system still prevails in London and in many other towns. Of late years—and principally by the influence and authority justly attached to the name of its most zealous advocate, Mr. Hawksley, beyond all dispute the ablest engineer practising the peculiar branch of the profession connected with the distribution of water—a system known by the name of "the constant and high pressure" has been introduced. It consists in so arranging the supply, that not only the mains but also the house services are always charged day and night, and the pressure is usually such as to insure the delivery of water at the highest level in a town at which it can possibly be required.

Abstractedly considered, there can be no doubt but that, in every point of view, the constant delivery must be the best. Any person who would take the trouble to look at, for it is not necessary to examine, the various receptacles (butts, tanks, or cisterns) used to contain water, in the poorer parts of towns especially, cannot fail to be disgusted with the foul contagion to which the water must be exposed.

In the houses of the rich, some precautions are taken to remove cisterns from the soot and filth of our town atmosphere, to place them beyond the immediate effects of the variations of temperature. But it is far otherwise with the houses of many of the middle and of all the poorer classes; and in them the recipients for the water required for domestic use are almost always placed in positions where they cannot fail to become corrupt, and to imbibe principles highly injurious to the hygienic condition of the unfortunate beings condemned to use them.

[Mr. G. F. Deacon, in 1875, forcibly pointed out the sources of waste of water in connection with the question of constant supply *versus* intermittent supply, and the evidence of the waste-water meter introduced by him. He divided waste of water into two classes: 1. Continuous or hidden waste, being that which flows from pipes or cisterns below ground, and sometimes by hidden pipes from cisterns above ground. 2. Discontinuous or superficial waste, being that which arises from defective fittings above ground, or from taps and valves temporarily left open. Out of every 100 gallons of water passing into a service main during twenty-four hours, says Mr. Deacon, it was not unusual (in Liverpool) for 35 gallons to be lost by continuous or hidden waste, and 35 gallons by discontinuous or superficial waste, whilst only 30 gallons were drawn off for use. The greater part of the waste is traceable to imperfection of fittings. Mr. Hawksley, speaking on this subject, found that the rate of consumption varied enormously in large cities where there was a constant supply day and night, and every person drawing as much or as little as he pleased. The consumption varied from 15 gallons to 100 gallons per head of the population per day, including the supply for manufacturing and sanitary purposes. No more water was wanted in the city where the quantity was 100 gallons per head per day, than in the town which was served

with 15 gallons per head per day. In many places the company or the corporation, or whoever might be the party supplying the water, merely turns the water into the pipes, and leaves the care of the internal fittings and the mode of their application entirely to the consumer, or to the builder, or to the landlord, as the case might be. The result is, as a rule, the worst possible character of fittings, and every cistern supplied with an overflow pipe, and where there is an overflow pipe, as a matter of course the ball-cock which lets in water is never attended to. The consequence is, the ball-cock gets out of order; it will not rise to shut off the water, and the water runs down the waste-pipe day and night. The same thing happens to soil-pans and also to water-closets, the handles of which are propped up under the notion of "doing good to the drains." The result is, the water runs away without anybody being sensible of the loss.

Mr. Hawksley logically stated the conditions of constant service contrasted with intermittent service, in the course of a discussion at the Institution of Civil Engineers in 1870.* The apparatus, he says, which has been adopted for rendering the constant supply of water successful by suppressing the waste, to which otherwise it would be subject, is distinct from the apparatus used for intermittent service. In constant service, pressure is applied to the pipes during the whole twenty-four hours instead of only during a very small portion of that time. Now if a service-pipe in the interior of a house would bear pressure for half an hour it would bear the same pressure for half a year; but, in the case of the constant supply, forces come into operation which do not actually operate in the intermittent system, or only to a small extent. For where the supply is intermittent, the draught during the short time the water is on, owing to the majority of the ball-cocks in the houses being open, very much diminishes the pressure, and besides that there are few or no shocks. But

* *Proceedings of the Institution of Civil Engineers*, vol. xxxi. p. 62.

upon the system of constant supply the pipes are subjected to all the shocks occasioned by the rapid closing of the cocks whereby the column of water is suddenly arrested when in rapid motion. That brings on a considerable amount of impulsive action which is unknown, or little known, in the case of an intermittent supply; and it is constantly found that the pipes leading into the houses and distributed through the houses, although perfectly competent to bear the pressure of the intermittent supply, will not bear that of the constant supply when it becomes introduced in place of the intermittent supply. From this it follows that, wherever the constant supply has been introduced, either voluntarily or by the pressure of the legislature, it has been found necessary to adopt rules and regulations for determining the magnitude and thickness of the pipes, also the mode in which the pipes should be united, and the kind of tap and ball-cock and water-closet apparatus to be used in connection with the constant pressure.

The rules and regulations, Mr. Hawksley proceeds to state, now found to be necessary, and very generally adopted, are reduced to writing. Formerly, universally, and still in most cases where the intermittent supply is used, a common plug-tap is applied. This has the effect, on rapid closing, of suddenly arresting the column of water in the lead service-pipe, and gives rise to two or three violent reactions. In time, the lead pipe expands into a sort of aneurism, and ultimately bursts by a long slit, exactly as an artery under similar circumstances bursts in the human body. Thus the introduction of a constant supply leads to the flooding of houses, damage of furniture, and destruction of property in many ways. But that has been entirely got over, and a cure established by the introduction of a screw-down cock in lieu of the old plug-tap. This closes slowly against the pressure of the water, and prevents recoil. Also, by reason of the looseness of the face, the leather which is interposed for the

purpose of making a perfect valve does not turn round on its face with the revolution of the screw; and, consequently, it is not ground or worn away as when the leather turns round with the screw. These valves do not lead to the bursting of the pipes, and are besides perfectly watertight, while other valves are not. Consequently the continuous trickle, which is often observed in other valves, and amounts to a serious quantity, does not occur. Moreover, the leathers can be replaced at about the cost of a penny, and so the cocks will last, with very little expense to the householder, for a considerable number of years. Probably, however, nine-tenths of the whole waste of water arises in the water-closets; and in every case where the constant supply has been attempted without a special apparatus to prevent the enormous waste which otherwise occurs in the water-closets, there has been a failure of the constant service system. In cases where there has not been total failure, the leakage has brought up the supply from under 20 gallons per head per diem to 50 gallons or more; and, in one town with which Mr. Hawksley is well acquainted, the amount of water distributed and wasted through water-closets has amounted to 110 gallons per head per diem. An apparatus has been arranged, and is largely in use, that has completely removed this difficulty. Water is introduced into a vessel in the usual way by means of a ball-cock. From this vessel, another vessel of a definite size, to hold one charge of water is filled through a communicating valve. When the wire is pulled, this, the first valve, closes, and a second valve opens and delivers the charge of water rapidly through a wide pipe into the basin to admit of a powerful flush of water. When the wire is released, the second valve is closed and the first valve is opened, and the second vessel is filled for another charge. At Norwich, where this apparatus has been in general use, the reduction of the expenditure of water has been from 40 gallons per head per diem to 15 gallons per head per diem.

Mr. A. R. Binnie* communicates the results of experimental tests of the access of pressure in lead pipes due to the sudden closing of the plug-tap on a moving body of water passing through the pipes. A lead pipe, ¾ inch in diameter, 114 feet in length, was connected at one end to a 3-inch supply main. At the other end there was fixed a plug-cock having an effective sectional area of ·152 square inches of waterway. The pressure of the water in the pipe was indicated by a gauge connected to the pipe near to the tap. The indicated pressures were as follows:—

Before opening	125 lbs. per square inch.
When open	20 lbs. ,,
When shut quickly . . .	550 lbs. ,,

The momentary exaltation of the pressure to 550 lbs. per square inch, amounted to nearly four and a half times the normal pressure, or the pressure at rest. But it was gradually reduced in successive oscillations to the normal pressure of 125 lbs. per square inch. The pressure-gauge was then removed to the other end of the lead pipe at the inlet, at a distance of 114 feet from the tap, when the indicated pressures were as follows:—

Before opening	125 lbs.
When open	120 lbs.
When shut quickly	220 lbs.

Mr. Binnie mentions, in contrast, two instances at Oxford and Cambridge, of economy of water realised under the constant service by the adoption of sufficient fittings; and waste of water, under intermittent service, with insufficient fittings. The water supply at Oxford under constant service, available to the consumer in any quantity, is consumed at the rate of just 12 gallons per head per day. On the contrary, at Cambridge, where the water supply is intermittent, lasting for 10 hours each day, the consumption and the waste together amount to 80 gallons per head per day.]

* "Chatham Lectures on Water Supply." Session 1877.

CHAPTER V.

POWER FOR RAISING WATER FOR WATER SUPPLY.

When the source of supply finally chosen is at a lower level than the points from which the distribution is to be effected, or than the highest point to which the water is to be delivered, it becomes necessary to employ some mechanical agent to raise it. For all town purposes the choice of the particular agent is limited either to steam or water power, according to the circumstances of the town under consideration; and in both cases the motive power must be applied to pumps, because they alone, of the various descriptions of intermediate machinery, can force the water to the height and the distance it is generally required to overcome.

The towns of Philadelphia and Richmond, in the United States, and of Toulouse, in France, are supplied by waterwheels, all undershot. Of these, the wheels at the Fairmount Waterworks, Philadelphia, are the most remarkable on account of the volume of water they are designed to lift. This is not less than about 10,000,000 gallons per day, with a dead lift of 92 feet, through cast-iron pipes 16 inches diameter. The engine house is built for eight wheels and pumps; the former being 16 feet diameter, 15 feet on the face, and with a fall of 7½ feet on the average, and making thirteen revolutions per minute. At Richmond there were two wheels 18 feet diameter, 10 feet on the face, with a 10-feet fall, working two pumps, and raising 800,000 gallons per day into reservoirs situated at a height of 160 feet above the low water.

At Toulouse, the wheels are 14 feet 5 inches in diameter, 5 feet on the face, with a fall of about 7 feet 6 inches; they are two in number, and raise about 896,000 gallons per day to a height of 67 feet above the water in the well.

In most of the large towns of England wherein it is necessary to employ mechanical power to raise the water, steam-engines are generally used, for the reasons before mentioned, or because the water power of the locality has been already appropriated. Until within a very recent period it was considered that when the power of the engine was required to exceed from 20 to 25 horses, the description of engine known as the Cornish engine was the most advantageous; but the results of the observations lately made upon the working of the double-action engines erected by Messrs. Simpson for the Chelsea and Lambeth Companies, and by Messrs. Bolton and Watts for the New River Company, would appear to reopen the controversy with respect to the merits of the various systems of pumping-engines. Below the limits above mentioned there is, however, a decided advantage in using the most direct-acting engines, both in respect to first cost and to subsequent working; or even in using small horizontal engines with fly-wheels, communicating motion to shafts bearing the pump-rods. In the case of the Cornish engines, or rather in any case wherein it may be necessary to raise large quantities of water to great heights, the most favourable conditions of movement in the pumps are, that they should begin by raising the load rapidly, and that when the first motion is perfectly determined, the effort used to move that load should be diminished progressively; so that, in fact, the motive power shall cease to act before the piston shall arrive at the end of the stroke. This is effected, in the Cornish engines, by introducing steam at great pressure upon the piston, through large orifices; the steam is then allowed to expand directly the inertia of the water has been overcome, and it has assumed an ascentional movement, which may be

maintained by a very small additional effort. In the engines used for raising water from the Cornish mines themselves, the initial pressure of the steam is about $2\frac{1}{2}$ to 3 atmospheres; the expansion begins at from $\frac{1}{8}$ to $\frac{1}{4}$ of the stroke of the piston; and at the end of the stroke the pressure is not more than from $\frac{1}{5}$ to $\frac{3}{10}$ of an atmosphere. In small pumping-engines, on the contrary, it is necessary that the action should be uniform; and on this account it is advisable to divide the action in such a manner as to work three pumps, by means of cranks, forming with one another angles of 120° upon the same shaft.

In the great London waterworks the style of engine usually adopted is the Cornish engine, and it may be worth while to mention here that in the East London Waterworks establishment the largest single machine of this description has been erected within a very few years, under the orders of Mr. C. Greaves. This engine has a cylinder of 100 inches in diameter, and 11 feet stroke, working a loaded pole of 4 feet 2 inches diameter with a velocity of 6 strokes per minute, and raising no less than 150 cubic feet of water in a stroke. But, as was said in the last paragraph, the experience of modern engineers appears to lean in favour of the use of beam and fly-wheel engines; and if all that is reported of the action of the machines erected for the New River and Lambeth Companies be correct, there would appear to be no doubt as to the superior efficiency of the principle upon which they are designed. The engines erected at the New River Head are of two kinds: four of them are double-cylinder engines, in which the high-pressure steam of the first cylinder acts expansively on the second, made by Messrs. Simpson and Co.; and the remaining two are single-cylinder engines, with a comparatively speaking small stroke, made by Messrs. Bolton and Watts. Many years since it was said that these machines were able to perform a duty equivalent to 98,000,000 lbs., raised 1 foot high, by the combustion of

1 cwt. of coal; and now the duty is said to be even carried so high as 120,000,000 lbs. Unfortunately, the experiments from which these results were obtained were not made contradictorily; and they must therefore be, for the present, received with caution. There is, however, one undoubted advantage possessed by the fly-wheel over the Cornish engines, viz. that they are capable of working at very different rates of delivery. In a town supply this may often become a matter of serious importance, as the demand is subject to very unexpected variations of an accidental nature; but, again, it must be observed that the working details of the latest pumping engines, of whatsoever description they may be, have been so carefully adjusted that they are even used without any regulating reservoirs or the old-fashioned stand-pipes so generally erected at the beginning of this century, and that the drivers of the engines can easily meet almost all the variable conditions of the consumption.

PUMPS.

[The merits of the rotative pumping-engine have become generally recognised for the purpose of water supply, more particularly in the form of the compound engine. At the same time, the direct-acting pumping-engine, with compound cylinders, has also made its footing in modern practice. Mr. Henry Davey has been very successful in the design and construction of his direct-acting compound engine and pump. Mr. Davey has effected two important improvements in pumping-engines, in which the length of the stroke and the velocity of the stroke are not controlled by a crank and fly-wheel. By the first improvement, the very high rate of speed which prevails at the commencement of the indoor stroke of the Cornish engine is obviated, whether the engine makes a large or a small number of strokes per minute, whilst at the same time a much greater average working speed of piston

is reached than in the Cornish engine. The second improvement consists in the prevention of "banging" in the event of a valve not closing, or of a pump-barrel bursting; for which purpose a "differential gear" is employed, by means of which the engine can be brought to a state of rest without any shock.]

CHAPTER VI.

MAINS AND DISTRIBUTING PIPES.

Between the pumping station or the collecting reservoirs of a water supply, and the point where the distribution to the various parts of the town commences, the water flows through a simple pipe of an uniform sectional area, and, as far as possible, with a constant, uniform velocity. In its course through pipes generally, however, the flow of the water is retarded by a series of resistances, which practically may be resolved into those depending—1, upon the friction on the sides of the pipes; 2, the loss of velocity occasioned by the bends; 3, the loss arising from the changes of direction from the mains to the submains or branches, if any such should exist; and 4, the gurgitation which occurs at every interruption in the flow.

The material usually employed for pipes is cast iron, and mains of 42 inches in diameter have been made of this metal. Very great precautions must be observed in laying pipes of such enormous diameter, and in regulating the pressure at intermediate points of their length; whilst it is also necessary to provide self-acting valves, or hand-valves, easily closed, in order to obviate as far as possible the chances of a rupture in the mains. Upon the Liverpool Waterworks, supplied by the great Rivington Pike reservoirs, Mr. Hawksley introduced some very skilfully devised machinery of this description; and in the Minutes of the Institution of Civil Engineers for 1859 will be found a tolerably clear account

of the valves and pressure regulators used upon the Melbourne Gravitation Waterworks. It is important also to observe that the pipes leading from a distant source of supply to a distributing reservoir must be placed (in fact, like all those required for a town distribution) at such a depth from the surface as to insure their being beyond the limits of the effects of atmospheric variations of temperature. In England, a depth of 4 feet is sufficient for this purpose; but both in extreme northern and southern latitudes it is necessary to descend considerably lower. In some portions of the distance between the ends of the mains it may likewise be advisable to insert double lines of pipes, and to make occasional connections between the two, in order that, in case of repairs to either of them, the flow may be maintained through the other.

[Eytelwein's formulas for the velocity and discharge of water in pipes are old-established formulas, which generally give satisfactory results. They are as follows :—

$$V = 50 \sqrt{\frac{dh}{l + 50d}}$$

$$Q = 2356 \sqrt{\frac{d^5 h}{l + 50d}}$$

V = the velocity of flow, in feet per second.
Q = the quantity discharged in cubic feet of water per minute.
d = the diameter of the pipe in feet.
h = the head or height of fall in feet.
l = the length of the pipe in feet.

Mr. James Simpson communicated the results of experiments on the flow of water through certain pipes, which confirmed in a remarkable manner the trustworthy character of Eytelwein's formula. The annexed table shows the observed or actual rate of discharge, together with the discharge calculated by that formula :—*

* *Proceedings of the Institution of Civil Engineers*, vol. xiv. p. 316.

Diameter and locality of pipe.		Length of pipe.	Head of water.	Actual discharge per minute.	Calculated discharge per minute.	Difference of discharges.
	Ins.	Feet.	Feet.	Cubic Ft.	Cubic Ft.	Per cent.
Main from Brixton to Streatham ..	12	5,200	38	205	200	— $2\frac{1}{2}$
	12	5,200	40	205	205	..
	12	5,200	$16\frac{3}{4}$	137	133	— 3
	12	5,200	19	137	141	+ 3
	12	5,200	4	$68\frac{1}{2}$	65	— $3\frac{1}{2}$
Main from Belvedere Road to Brixton	19	22,440	41	323	325	+ $\frac{1}{2}$
	19	22,440	$43\frac{1}{2}$	331	338	+ 2
	19	22,440	34	298	304	+ 2
	19	22,440	$27\frac{1}{2}$	267	272	+ $1\frac{7}{8}$
	19	22,440	24	243	247	+ $1\frac{5}{8}$
Main at Liverpool	12	8,140	27	$124\frac{1}{2}$	134	+ $7\frac{1}{2}$
	12	8,140	$24\frac{3}{4}$	119	127	+ $6\frac{3}{4}$
	12	8,140	18	104	110	+ $5\frac{3}{4}$
	12	8,140	12	89	89	..
	12	8,140	$5\frac{1}{2}$	67	61	— 9
Main at Carlisle ..	12	6,600	$34\frac{1}{2}$	168	167	— $\frac{1}{2}$
	12	6,600	46	189	195	+ 3
Main from Ditton to Brixton......	30	54,120	25	521	510	— 2

Mr. Hawksley's general formula for the velocity of flow of water through pipes, is as follows :—

$$V = \cdot 77 \sqrt{\frac{hd}{l + 2\frac{1}{4}d}}$$

V = the velocity, in yards per second.
l = the length of the pipe, in yards.
d = the diameter of the pipe, in inches.
h = the head, in inches.

The same engineer employs the following formula for the velocity of water in a smooth pipe of small and uniform diameter :—

$$V = 48 \sqrt{\frac{h}{l} \times d}$$

V = the velocity in feet per second.
l = the length of the pipe in feet.
d = the diameter of the pipe in feet.
h = the head in feet.]

The formula which expresses the conditions of the flow of water in a pipe of uniform diameter, and working under a constant pressure, ceases to be applicable when there is a series of side branches or of submains, deriving their supply immediately from the principal one. During the course of the distribution, a difference in the volume of water passing through the pipes must necessarily arise from the mere fact that a portion of the water will be drawn off by the side mains; and therefore in the latter parts of their course, the supply mains must be proportionally diminished to the service they are designed to supply. But it may sometimes happen in practice, that the cost of new models for smaller pipes may be so great as to render it more economical to retain the original dimensions of the mains; so that this question of detail must be carefully considered in forming the comparative estimates of the various modes of effecting the supply. It is, however, always necessary, before deciding the dimensions of any main pipe, to take into account not only the absolute theoretical requirements of the case, but also the probability of any eventual increase in the supply which the mains may have to carry.

Mr. Hawksley stated that the method he adopted to ascertain the diameters to be given to the pipes laid down upon what is called the constant delivery system (in which the pipes are always under charge, and no cisterns are used), is to divide the length of the main in a street into portions of 200 yards each, and to assign to every such portion the quantity of water it would be likely to require, on the supposition that that quantity would be discharged in four hours. He then allows for a loss of head equal to 4 feet in every 200 yards, and adopts, in calculating the diameters to be given, the formula $\frac{1}{15}\sqrt[5]{\frac{q^2 l}{h}} = d$, in which $q =$ the number of gallons; $l =$ the length of the main in yards; $h =$ the head in feet; and $d =$ the diameter required, in inches.

The pipes from the pumping stations to the distributing reservoir (and, generally, all pipes required for a town distribution) should be laid about 4 feet below the surface, and carefully covered with earth and sand, or some non-conducting materials. The object of this precaution is to protect them against the effects of frost, to maintain an equal temperature in the waters, and to place them beyond reach of injury from shock or jar by passing weights. In some portions of the distance between the two stations it may also be advisable to insert double lines of pipes, and to make occasional connections between the two, in order that, in case of repairs to either of them, the flow may be maintained through the other.

These remarks have necessarily been confined to the consideration of the cases in which water is raised from a lower level and pumped through pipes. If the source, however, be situated at a distance, and at a higher level than the commencement of the distribution, the course to be adopted must necessarily be modified. In such cases, if no very serious obstacles are to be met with, it is preferable that the water be led in a conduit rather than in a pipe; for evidently the friction in the latter is much greater, and the height of the point of arrival diminished in proportion. Such conduits should be covered in all situations where the quality of the water is likely to be affected, as in the neighbourhood of large towns, or during their passage through forests and underwood; or, again, in warm climates, where the temperature not only acts injuriously upon the quality, but also gives rise to an evaporation of a very serious character. But if the conduits be so covered, there must still be adopted precautions for insuring a perfect ventilation and occasional renewal of the air; and in the Roman aqueducts, wells also were formed at occasional intervals, to allow of the deposition of any matters in suspension.

CHAPTER VII.

CONDUITS, AQUEDUCTS, TUNNELS, FOR WATER SUPPLY.

An advantage of great importance attached to the use of conduits, rather than of pipes, for the conveyance of spring waters, lies in this, that any of the earthy salts in solution which they are likely to deposit in the course of time, are not so likely to produce injurious effects in open culverts as they are in close pipes. The separation takes place, in air, at an earlier period of the flow; and it must evidently be more easy to cleanse or repair such conduits than it can be to perform the same operation upon pipes buried in the ground. On the other hand, it must be admitted that the construction of a small conduit is always a more expensive operation than the employment of pipes would be to insure the discharge of the same volume of water; so that, eventually, considerations of economy may outweigh those derived from the theoretical advantages above cited. As an illustration of the extent to which the deposition of the earthy salts may interfere with and contract the effective area of a watercourse, the accompanying sketch of the transverse section of the conduit upon the celebrated aqueduct of the Pont du Gard is added. The portion shaded of a darker colour, round the watercourse, represents the deposit of

Fig. 277.—Conduit Pont du Gard.

calcareous matter which has gradually accumulated by precipitation from the waters, although great pains had previously been taken to insure their purity.

The most serious difficulties which are likely to be encountered in the construction of conduits, are those arising from the occurrence of hills or deep valleys in the line they ought to follow. The former, if of considerable elevation, will require to be traversed in tunnel; the latter may be passed either by aqueducts, or by syphons descending from a reservoir on one side, and remounting to a second at a lower level, on the other; the conduit recommences from the second reservoir.

It was the dread of the deposit from the waters of the Durance which induced M. de Montricher to adopt the mode of conducting them to Marseilles, which he finally did; and he was induced by the same motive to construct that splendid folly, the aqueduct of Roquefavour, shown in Fig. 278. At the Liverpool Waterworks Mr. Hawksley adopted the less showy system of subterranean pipes, which, it may be added, is usually followed by English engineers; for, working as they almost always do for commercial companies, they are not often allowed to indulge the fancy for erecting comparatively useless monuments.

The dimensions and form to be given to tunnels must necessarily be regulated, so far as the minimum is concerned, by the consideration that the workmen must be able to use the various tools, and to push to the extraction pits the materials disengaged during their operations. The nature of the rocks traversed will also affect the sectional area of the excavation; for if it be of a nature to render lining indispensable upon the sides and top, as well as for the water channel itself, the dimensions evidently must be increased. A miner can work with tolerable efficiency in a heading of

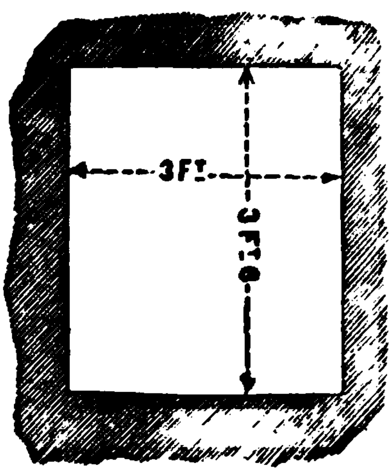


Fig. 279.—Heading.

the size represented in Fig. 279; but it must be considered as the minimum in all cases, because the constrained position of the workmen prevents their employing the whole of their useful power, and below this size they could hardly advance themselves, without at all being able to work. It is also important that the workmen should be able at any time to visit and repair every portion of the tunnel. For these reasons the conduit from which all the mains for the supply

Fig. 280. Aqueducts in Paris. Fig. 281.

of Paris draw the water is made of the dimensions indicated in Fig. 280 above. But it is also to be observed that this conduit, called the "Aqueduc de Ceinture," is about $1\frac{6}{10}$ mile in length, and has only a fall of 4 inches throughout, so that the flow of the water only takes place in consequence of the difference of level caused by the withdrawal of the water through the various pipes branching from it. The section is therefore much larger than it would be otherwise; and perhaps the desire to make it sufficient for the passage of a boat, hauled by a man upon the species of towing path, may have led to some exaggeration of its dimensions. Fig. 281, represents the section of the branch "Aqueduc St. Laurent," joining the "Aqueduc de Ceinture," and supplying one of the quarters of Paris.

CHAPTER VIII.

GLASGOW CORPORATION WATERWORKS.

[A TYPICAL example of waterworks in which the water is supplied by gravitation, is supplied in the case of the water supply of Glasgow. Here, excellent examples of aqueducts, bridge aqueducts, and syphon-pipes for crossing valleys, are supplied. The following is a general account of these works.*

The Glasgow Corporation Waterworks—Loch Katrine Waterworks—were designed to supply pure water to the city of Glasgow from the system of lochs comprising Loch Katrine, Loch Achray, Loch Drunkie, and Loch Vennachar. Of these, Loch Achray has not been interfered with. The works were designed by and carried out under Mr. J. F. Bateman, as engineer. They were commenced in 1856, and completed in 1859. The works at the outlet of Loch Vennachar consist of a dam of masonry across the mouth of the loch, and a new channel for the river, to enable the water to be drawn down below the old summer level. The new channel is 700 yards long and 50 feet wide. At the lower end, the compensation gauge-weir, 100 feet wide, is placed. It is formed by a continuous cast-iron plate brought to a thin edge at the top. At the upper end of the channel, next to the loch, a range of cast-iron sluices is built into a block of masonry 110 feet long and 15 feet thick, with eleven

* Derived from Mr. J. M. Gale's paper on the "Glasgow Waterworks," in the *Transactions of the Institution of Engineers in Scotland*, 1864, vol. vii. p. 21; and from Simms's "Practical Tunnelling," third edition, by D. K. Clark, 1877, p. 232.

arched openings for discharging the water. Three of the sluices have a clear width of 4 feet, and a height of 4 feet. Four of them are 6 feet wide and 2 feet high; and the remaining four are at the upper end of salmon stairs, formed to allow the fish to get into the loch at its different levels. The salmon stairs are each 6 feet wide below the walls, and have a general inclination of 1 in 12. These sloping channels are formed into a succession of deep pools by means of planks on edge placed across the channel, over which the water falls. The height of the plank is varied as the level of the water in the loch changes, so as to keep a constant overflow of a depth of from 15 to 20 inches. The top of the dam is roofed over, forming a sluice-house for protecting the working gear. The waste-weir of the loch is 150 feet wide, and is a continuation of the masonry of the dam across the top of the new river channel. The raising of the level of the loch involved the diversion of several roads. At the upper end, where the ground is level, about 150 acres of meadow land, known as Lanrick Mead, are under water when the loch is full. The cost of these works amounted to about £26,000.

Loch Drunkie has been raised 25 feet in level by means of two earthen embankments, puddled in the usual manner, whilst the area of the loch has been increased by about 60 acres. The northerly embankment is 150 yards long and 21 feet high. The other embankment, which was constructed at the original outlet of the loch, is 40 yards long and 32 feet in height. A cast-iron pipe, 24 inches in diameter, is laid through this embankment, fitted with a valve at the outer end, to regulate the rate of discharge.

The works at the outlet of Loch Katrine are similar to those at Loch Vennachar, but they are on a much smaller scale, as the quantity of water to be discharged from that loch is less than from the other. There are two sluices 4 feet wide and 4 feet high, and two salmon stairs 6 feet wide in the masonry dam, and a waste-weir 100 feet in length.

The point at which the aqueduct leaves the loch is about five miles from the outlet. The basin at the inlet to the aqueduct is 55 feet long by 40 feet wide inside; it is constructed with three iron sluices, each 4 feet square, for regulating the flow in the aqueduct, and a line of strainers across the middle to prevent fish and other objects from passing into the aqueduct.

The aqueduct is 25¾ miles in length—that is to say, from Loch Katrine to Mugdock reservoir. Of this length, 13 miles consist of tunnelling, 3¾ miles are iron piping across valleys, and the remaining 9 miles are of open cutting and bridges. The tunnelling comprises eighty separate tunnels, for the construction of which 44 shafts were sunk. For the first ten miles, the rock consists of mica-schist and clay-slate —close retentive material, into which no water percolates, and in which, consequently, few springs were to be found. "This rock," says Mr. Bateman, "when quarried, was unfit for building purposes; there was no stone of a suitable description to be had at any reasonable cost or distance, no lime for mortar, no clay for puddle, and no roads to convey material. Ordinary surface construction was, therefore, out of the question. . . . The aqueduct may be considered as one continuous tunnel. As long as the work continued in the primary geological measures, we had no water; and even after it entered the Old Red Sandstone, and when it subsequently passed through trap-rock, there was much less than I expected."

The tunnels were constructed with a flat floor, vertical sides, and a semicircular arch. They are 8 feet wide, and 8 feet high at the centre, except in loose rock, which was not water-tight, when the arch and the sides were formed slightly elliptical, to a width of 8½ feet, with an invert. The three types of section, according to which the tunnel was constructed, are shown in Figs. 282, 283, and 284, of which the first, simply excavation, is formed in solid water-tight rock; the

Fig. 282.

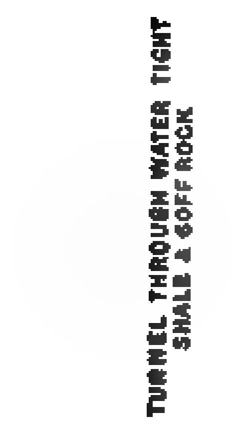


Fig. 283.

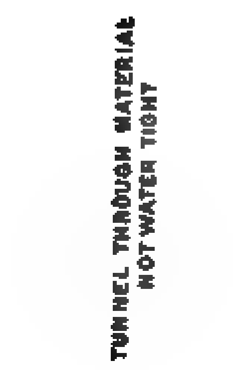


Fig. 284.

Glasgow Corporation Waterworks: Tunnels.

second, through water-tight shale and soft rock, with 15-inch walls in rubble and mortar, and 9-inch arch parpoints set in mortar, packed behind with dry rubble; the third, through soft rock or shale, not water-tight, with 9-inch brickwork in mortar throughout, an extra half-brick thickness at the sides, and dry rubble packing above the arch.

The aqueduct begins with the first tunnel, which abuts on Loch Katrine, and is cut through the ridge which separates Loch Katrine from Loch Chon Valley, consisting of clay-slate and mica-schist, with beds of gneiss, mixed with quartz. The tunnel is 2,325 yards long, and is 500 feet below the level of the summit of the ridge. Twelve shafts were sunk from the surface, varying in depth from 14 yards to 163½ yards, making an aggregate length of 1,173 yards, or half the length of the tunnel. Their average distance apart was about 200 yards. The entrance is shown in elevation and longitudinal section in Figs. 285.

The next important tunnel is the Clashmore tunnel, through a ridge of Old Red Sandstone conglomerate, 1,175 yards in length; for the construction of which three shafts were sunk, respectively 53, 22½, and 29 yards deep.

The aqueduct terminated in the Mugdock tunnel, 2,640 yards long, through a ridge of amygdaloidal trap, in which seven shafts were sunk, averaging about 130 yards apart.

Several portions of the tunnels were lined with brick. In some places, the rock, which was at first considered durable, was found to perish after exposure to the atmosphere: the Old Red Sandstone was, for this reason, lined for a considerable extent.

The rock was drilled for blasting by hand labour. It proved to be extremely hard and difficult to work, especially the mica-slate. In many cases, in cutting the tunnel through the rock, the progress did not exceed 3 linear yards per month at each face, though the work was carried on day and night. The average advance in the mica-slate was about

5 yards per month, or 7 inches per day. The boreholes were 1¼ inch in diameter, and their ordinary depth was

SECTION OF ENTRANCE TO TUNNEL

ELEVATION OF ENTRANCE TO TUNNEL

Fig. 285.
Glasgow Corporation Waterworks.

20 inches. The drills required to be removed at every inch of depth, and the time occupied in drilling a hole was

about an hour and three-quarters, equivalent to $\frac{1}{8}$ inch per minute. A gang of men, working at an 8-feet face, started at night, with a supply of 60 drills or jumpers, and brought out 60 dulled drills in the morning. As there were often 100 faces wrought at once, hand labour was considered preferable to perforating machines for the work. The junctions, upwards of 200 in number, were exact, and could only be traced afterwards by the crossing of the drill-holes blown out.

The ruling gradient, or fall, of the aqueduct is 10 inches per mile, equivalent to 1 in 6,336; and the aqueduct is capable of passing 50,000,000 gallons per day.

The actual cost of removing the rock by blasting varied from £1 to £2 per cubic yard. The cost of tunnelling through the mica-slate was about £13 per lineal yard, and through the clay-slate from £9 to £10 per lineal yard. In the Old Red Sandstone, when the discharged water was so considerable as to retard the progress, the cost in the lower beds of the stratum was about £10 per lineal yard. Through the softer strata, the cost of excavation was £8 per lineal yard, and in some cases even less than that. These costs include the cost of the shafts, except where they were of considerable depth, as in the Loch Katrine tunnel, for which the cost of the shafts is not included. The cost of driving through soft material, and of lining, taken together, was about equal to the cost of excavation through hard and compact rock, when no lining was required.

The aqueduct bridges over the ravines are somewhat peculiar. There are five of considerable length, respectively 124, 154, 212, 147, and 332 yards in length, similarly constructed, as in Fig. 286. At the ends of the bridges, or the shallowest parts of the ravines, the aqueduct is a cast-iron trough, Fig. 287, 8 feet wide and 4 feet deep, of $\frac{5}{8}$-inch plates, supported on a solid dry-stone embankment, constructed of the stone of the district carefully set by hand, 9 feet wide at the

top, with a batter of 3 inches to a foot at each side. The deeper parts of the valleys are crossed by cast-iron tubes, Fig. 288, 8 feet wide by $6\frac{1}{2}$ feet high inside, supported on piers at intervals of 50 feet. The bottoms and sides are of $\frac{5}{8}$-inch plates, and the tops are 7-16 inch plate stiffened by angle irons. The level of the bottom of the tubes is 3 feet below that of the troughs. The tubes can pass 50,000,000 gallons per day. At the crossing

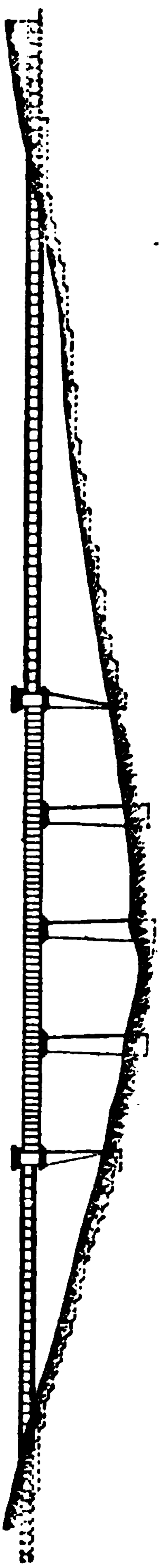


Fig. 286.—Iron Aqueduct Bridge, Glasgow Corporation Waterworks.

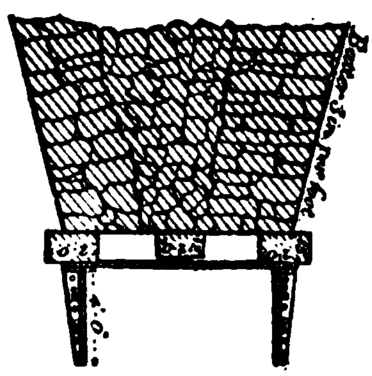


Fig. 287.

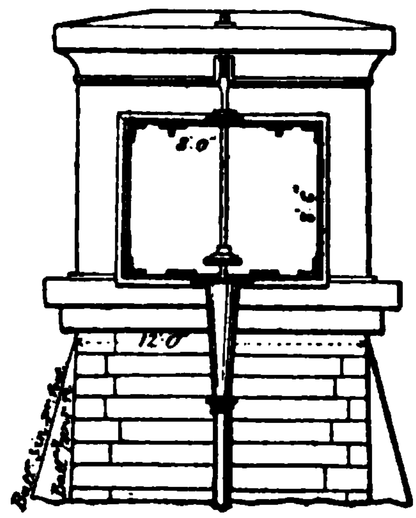


Fig. 288.

of small mountain streams, the aqueduct is carried in cast-iron troughs like those already described, supported on cast-iron beams over the stream.

Siphon pipes are laid to cross the valley of the Duchray Water, about 1,210 yards wide. Small basins are formed at each end, from which the pipes proceed. These are 4 feet in diameter, in 9-feet lengths, with spigot and faucet joints, run in with lead in the usual way. At the lowest point, the pipes are under a pressure of 165 feet. The river is crossed by cast-iron girders of 60 feet span. Provision was made for laying two additional lines of pipes, one of 4 feet and the other of 3 feet in diameter.

After passing through the ridge of Old Red Sandstone con-

glomerate by the Clashmore tunnel, the aqueduct is, for a length of five miles, in open cutting for the greater part, with masonry sides and a dry-rubble arch, covered with puddle 2 feet in depth.

The Endrick Valley, like that of the Duchray, is crossed by a 4-feet siphon pipe, $2\frac{1}{4}$ miles in length, and is subject to a pressure of 235 feet of water at the bottom of the river, where the pipes are $1\frac{3}{8}$ inches in thickness. The pipes are carried across small depressions in the valley on stone piers, and at the crossing of two roads, and of the Forth and Clyde Railway, they are further supported by cast-iron brackets. At such exposed places the joints of the pipes are flanged. There is a short tunnel on this length of pipe sufficiently wide to carry the three lines of pipes.

Fig. 289.—Stone Aqueduct Bridge, Glasgow Corporation Waterworks.

The construction of the aqueduct for the five miles extending from the valley of the Endrick to the valley of the Blane presents the same general features as those already described. Good building stone was abundant in this district, and the bridges are all of masonry. One of the aqueduct bridges near Killearn, nineteen miles from Loch Katrine, is represented in Fig. 289.

The total cost of the aqueduct was £468,000, averaging £18,000 per mile.

The 4-feet pipes were intended to deliver 20,000,000 gallons per day. They have, in fact, delivered 24,000,000 gallons per day, and even then they were not completely charged. All the pipes were coated with coal-pitch and oil, according to the process of Dr. Angus Smith, first applied by Mr. Bateman for

the Manchester Waterworks. This coating, when properly done, imparts a smooth glassy surface to the pipes, and prevents, at least for a number of years, oxidation of the metal.

At the Mugdock reservoir, the water is first discharged into a basin, from which it passes over cast-iron gauge-plates, 40 feet wide, brought up to a thin edge. The depth of water passing over these plates is regularly recorded, and the discharge computed. From the basin, the water falls into an upper division of the main reservoir, about two acres in extent, and thence it is discharged into the main body of the reservoir.

The reservoir has a water surface of 60 acres, and a depth, when full, of 50 feet. It contains 548,000,000 gallons, and is 317 feet above ordnance datum. The reservoir was formed by means of two earthen embankments, of which the principal embankment, shown in section, Fig. 290, is 400 yards long and 68 feet high; and the other, or easterly, embankment is 240 yards long and 50 feet high. Each embankment is made with a central puddle wall, and is pitched on the front slope.

The water is drawn from the reservoir by pipes laid in a tunnel through the hill between the two embankments. At the inner end a stand-pipe is arranged so that water can be drawn at various heights.

Fig. 290.—Mugdock Reservoir, Glasgow Corporation Waterworks.

About 50 yards from the reservoir, the water passes into a circular well cut out of the rock, 40 feet in diameter and 63 feet deep, where it is strained through copper-wire cloth. The wire-cloth contains 40 meshes to the inch, arranged in oak frames, forming an inner well,

octagonal in shape, 25 feet in diameter, from which the supply is delivered into the lines of pipes leading to the city.

The two pipes are each 42 inches in diameter, until they emerge from the tunnel, where they are reduced to a diameter of 36 inches. They are continuous at this diameter to the city. They are laid side by side for a length of three miles, after which they diverge, one line being carried by the Great Western Road for the supply of the low-lying districts, and the other by Maryhill for the supply of the high districts. The pipes meet again at St. George's Road, and are so arranged as to be put in connection when required. To this point, the distance from the straining-well is 7 miles for the low pipe, and 6½ miles for the high pipe. Each line of pipe crosses the river Kelvin and the Forth and Clyde Canal. The Kelvin Bridge, on the Great Western Road, was widened to carry, on cast-iron girders, the low district main.

At Mugdock reservoir, self-acting closing valves, intended to shut off the water on the occasion of a pipe bursting, are attached to each line of pipes. Stop-valves are fixed at intervals along the line of mains, both in the country and in the city. On the side next the reservoir, at each stop-valve, a momentum-valve is applied, designed to prevent concussion in the pipes by the too sudden closing of the stop-valves. At the summit of each rising ground, on pipes of 6 inches in diameter and upwards, an air-valve is fixed; and at the bottom of every hollow a flushing-out cock is attached, by which the pipes may be emptied for repair. Man-holes are placed at intervals along the lines of the large mains, and close to the large valves, to afford admission for inspection or for making repairs.

The arrangement of the self-acting closing-valve is of the character of a throttle-valve fixed across the pipe. It is held open when the water passes at a given velocity, by means of counterbalance weights. When the velocity is exceeded from any cause, as the bursting of a pipe, the valve closes.

That the large stop-valves might be worked easily by one man, they were divided into compartments, on a principle proposed by Sir W. G. Armstrong, so subdivided that one compartment-valve could be easily worked at a time by one man. When the smaller division is first opened, the passage of water through the opening so much relieves the pressure on the slide of the larger compartment, that it also can be opened with ease. The total area of opening through the valve is something less than the sectional area of the pipe; and it requires 1 foot head of pressure to pass the quantity of water that the pipe was designed to deliver. Valves of a diameter greater than 16 inches are of this construction. For 36-inch pipes, of which the sectional area is 7 square feet, the clear water way is 4½ square feet, and the smaller slide has an area of 1 square foot, that of the larger slide being 3½ square feet. The water passes through the contraction at the speed of 6¾ feet per second, to correspond with the normal velocity of 4·4 feet per second in the body of the pipe.

The consumption of water per head per day in Glasgow, amounted, in 1838, to 26 gallons; in 1845, to 30 gallons; in 1852, to 35 gallons on the north side, and 38 gallons on the south side. In 1863 the consumption reached to 42½ gallons per head per day. It was considered that, of this amount, 15 gallons was run to waste.

The total cost of the works was as follows:—

Works at the Lochs	£36,000	
The Aqueduct, 25¾ miles in length	468,000	
Mugdock Reservoir	56,000	
Main pipes, 36 inches in diameter	123,000	
Distribution in the city	78,000	
Total for works		£761,000
Land and compensation	70,000	
Parliamentary expenses, engineering, and sundries	87,000	
		157,000
Total		£918,000]

CHAPTER IX.

DRAINAGE OF LAND.

The functions of vegetable life cannot be carried on without the presence of a certain quantity of water, inasmuch as the fluids which circulate in their tissues are almost entirely composed of the water taken up by the roots from the ground. With the exception, however, of some aquatic plants, the majority suffer from an excess of humidity; and when water is found in an agricultural district in large quantities, it is as injurious as its absence is in other cases. Thence arises the necessity for *draining* lands surcharged with water, on the one hand, and for *irrigation* on the other. It is equally important that air should be allowed access to the roots of plants; but the operation of ploughing, harrowing, hoeing, &c., by which this object is effected, belong to the science of agriculture rather than to engineering.

The nature of the surface and of the subsoils produce effects upon the humidity of a district which are more readily under control than the causes previously alluded to. They act either by retaining the surface water, or by giving passage to the springs fed by lands at a greater distance; and it is of the utmost importance to be able to distinguish between these two sources of humidity, as the surface drainage adapted to the first, under some circumstances, is utterly ineffectual to remedy the second.

For drainage operations, the strictly correct geological descriptions of the various strata may be neglected, and they

may be divided simply into two classes, the *porous* and the *impervious*. The former comprises all those consisting of loose materials which absorb water easily and allow of its passing freely, such as gravel, sand, loamy clays, and the comminuted upper strata of most of the limestone formations. The latter consists of stiff blue clays or of the plastic clays found in such abundance; of some kinds of gravel cemented by argillaceous, calcareous, or ferruginous materials; and of such limestone, sandstone, or granitic rocks as present a close grain without any fissures. No regular order of superposition of these classes of strata exists in nature, and from their complication arise the greatest difficulties in drainage.

In such cases as those in which a pervious stratum lies upon an impervious one, the water falling from the clouds permeates the former until it meets the latter. If, then, no escape be furnished by some natural overflow, the water must accumulate in the lowest depressions, until the hydrostatic pressure of that in the higher portions forces it to the surface in any lower ones whose conditions of level may be such as to allow of its rising. It may frequently happen that a natural overflow exists at a small distance from the surface, but not at such a depth as to prevent the existence of great moisture in the main body of the stratum, although no external indication beyond the character of the herbage may indicate the moisture. The great objects, therefore, in all drainage are, not only to remove the surface waters, but more particularly to cut off the subterraneous waters, which either rise to the surface or are confined beneath it.

The removal of surface waters is a comparatively simple operation, for it may be effected by dressing the land into ridges, and giving these an outfall into a drain or ditch all round the field. The ditch itself would pour its waters into any natural course, and the latter may at any time be enlarged or improved by observing the principles regulating the flow of water in open channels already

laid down. The conditions to be observed being, that the channel should be able to carry off, at a suitable velocity, the maximum quantity of water likely to be thrown into it within a definite period; and that the velocity should not be such as to endanger the bottom or the sides. If the outfall drain be artificially made, it is, generally speaking, desirable that it should be impermeable.

Operations connected with the improvement of an outfall affect very large areas, and would seem almost to call for some action of the Legislature. In many individual cases, so to speak, it is beyond the power of one proprietor to undertake them; and the only course left open to him is, to isolate his own land by diverting any water flowing from other districts, and to remove that which falls upon his own, by means the most adapted to effect that object economically. The execution of an intercepting drain will very frequently suffice to remove all the subterranean waters, should such be found, by stopping the flow of the latter in what would otherwise be their natural direction, and thus leave merely the rain-water falling over the particular district to be dealt with. In such countries as Holland, and the fens of Lincolnshire, Bedfordshire, &c., the intercepting drain itself becomes the outfall and a means of communication; for the main drains are used as canals, and the waters from the low lands are pumped into them either by windmills or by steam power, as may be most expedient.

In hilly countries it rarely happens that any difficulty occurs from the direction or inclination of the watercourses, and in them the question of outfall is not so complicated as in the lower and more level districts near the embouchures of rivers. The longitudinal section of the centre line of nearly all rivers is, in fact, a concave parabolic curve, the apex of which is in the elevated grounds near its source. The velocity, under such circumstances, is very great in hilly countries, and the streams are able to keep their course

in a tolerably straight line, if even they do not continually tend to rectify any bends which may naturally exist. But in proportion as the rivers approach the sea, or other large rivers, they usually flow through flat alluvial deposits, or through level plains of earlier formations. The velocity of the water diminishes, and the gradual deposition of matters brought down from the hills raises the bed of the river, whilst the direction becomes tortuous from the incapacity of the stream to overcome the obstacles to its progress. In no country in the world can more striking illustrations of these laws be found than in England; nor, perhaps, is there any country where well-directed works for the purpose of obviating their inconveniences would be attended with more brilliant results.

Before commencing any rectification of the bed of a river or stream, it is necessary to inquire carefully into all the numerous commercial interests which are likely to be affected by the alteration. A plan of the existing watercourse and its various affluents, with longitudinal and transverse sections of the beds and banks to a considerable distance on either side, is required; observations upon the flood and summer levels, and upon the seasons and durations of the changes in the volume of the stream, must be made; and, lastly, a careful notice must be taken of the nature of the materials carried down, the mode in which shoals are formed or the banks destroyed, and the nature of the river-bed in its normal state.

If the stream follow a very tortuous course, a new channel in a direct line evidently will shorten the distance between its extreme points, and increase the inclination of the water line. The velocity of the stream will be proportionally augmented, and if the same quantity to be discharged flow before and after the execution of the new channel, its sectional area may be made smaller; or if, on the contrary, it be made of the same area as the original channel, it will be

able to discharge a greater volume. Any sudden bends may thus be avoided; but it is to be observed, that there seems to exist some law, the cause of which has hitherto escaped our analysis, owing to which rivers are not able to flow in straight lines for any great distance, in other than beds of masonry, without requiring great and frequent repairs. At any rate, every stream when left to itself, so to speak, assumes a tortuous outline; and, from the experience obtained in France and Italy, it appears that after a deviation there is always a tendency to resume the original directions, especially during the seasons of floods. It is, therefore, preferable that the centre line of a new channel be formed with a series of curvatures of very large radius rather than in a perfectly straight line. Upon the Rhine it was found that the river exercised no corrosive action upon its banks when the radius of curvature was about 2,750 yards long, the bed of the river consisting of sand and gravel, and being frequently exposed to sudden and violent floods.

The efficient action of new channels can only be attained by observing these conditions:—Firstly.—They must be deepened as much as possible; the sectional area to be given will of course be regulated by the volume to be discharged under all the varying conditions of the rain-fall. Secondly.—They must not present any sudden projections, or form any sharp curves with the main stream. Thirdly.—If the new channel cannot be dug out at once to the required depth, it must not be opened to receive the waters until the *down* stream end of the old channel be closed, so as effectually to force all the running water into the new channel. Fourthly.—All obstacles, such as trunks of trees, large blocks of stone, &c., must be removed, so as to leave the watercourse perfectly clear.

When an entirely new outfall is to be formed, the dimensions to be given to it must depend upon the proportion of the rain-fall it may be required to carry off. This will vary,

not only according to the configuration of the country, but also according to the greater or less degree of permeability of the materials. In precipitous mountain districts the rain flows off with comparative rapidity, merely from the inclination of the ground. Should, however, our observations be directed to particular mountain districts, it will be found that the discharge from granitic rocks differs very materially from that from the lias, the oolites, or the clay formations. From the granites, the rain runs off nearly as fast as it falls, for the materials are non-absorbent, and the subordinate outlines do not present any depressions likely to retain the water. The lias is also, comparatively speaking, impermeable, as are also the clays; whilst the oolites and the gravels absorb the water during the period of its falling, to give it out again when perhaps the supply may have ceased. In fact, the character of the discharge from the granites, the lias, and the clays may be regarded as being of a torrential description, whilst that from the limestones is far more equable. In the former districts, it appears that about $\frac{2}{3}$ of the rain flows off in the natural watercourses, whilst in the latter and in the gravel the maximum quantity so flowing would only be $\frac{1}{3}$. Again, the proportion of the rain-fall which may require to be carried off will differ, according to the greater or less continuance of the rainy season. Thus in winter it happens that the ground frequently becomes saturated with water at an early period, and it is advisable in such a case that any flood should be carried off as rapidly as it rises. The maximum quantity of rain which may fall within a given time becomes then a condition regulating the dimension of the outfall, of nearly as much importance as the average fall of the whole year.

An outfall having been secured, either by adopting or improving the natural facilities of the country, or by forming a new watercourse, if the source of the water deteriorating the quality of any land be not such as to be removed by

surface drainage, an investigation of the surrounding district must be made, to ascertain the superposition of the strata, their nature, thickness, and respective inclinations; or, should any local circumstances prevent this examination from being carried out on a sufficiently extended scale, small ditches or trial shafts should be sunk at the upper and lower sides of the district to be drained. The points of outburst of any springs must be noticed, and, if possible, their sources of supply be discovered. When these points are settled, the direction to be given to the drains must be considered; and, if possible, it would be advisable to make them follow the line of the longest fall of the ground. The depth, and the distance apart of the drains, must depend to a certain extent upon the description of crops to be raised, but more particularly upon the nature of the subsoil. For, in the first place, it is necessary to place the drains at such a depth as to obviate any danger of their materials being deranged by agricultural operations. In ordinary modes of cultivation, the minimum depth to which the ground is worked may be taken at 8 inches; in many others, the ground is moved to a depth of 18 inches; and for these reasons it is usual to place the drains at such a depth that there shall be a distance of about 20 inches between their highest points and the surface of the ground. In the second place, if an impermeable subsoil be met with within a distance of 5 or 6 feet from the surface, such as to intercept the passage of the water in either direction, the drains must be carried down to it; or otherwise the portions between each of them would only be imperfectly dried. The nature of the materials employed will also modify the depth of the drains; for if they be bulky, as in the case of broken stone, they must require a greater width than when tiles or tubes are used.

The width of the trenches will be regulated by the depth of the drains, because the workmen require a greater space to work the deep than they do the shallow ones. At the

surface the width is required to be greater than at the bottom; and in practice it is found that, for a depth of about 3 feet, it is sufficient to give a width of about 1 foot at the surface and of 6 inches at the bottom; for a depth of about 4 feet, those dimensions become respectively 1 foot 4 inches and 8 inches; whilst, for a depth of 8 feet, they become respectively 2 feet 6 inches and 1 foot 2 inches. The direction of the drains should be made as straight as possible, in order to avoid any interference with the discharge of the water; and they must be commenced by opening the lower portions of the district first.

It is indispensable that a regular inclination be given, and that it should be sufficient to insure the flow of the water. A fall of about 1 in 200 will be found sufficient for ordinary cases, especially if the drain tiles be well laid.

There are several modes of filling in drains employed by agricultural engineers, the principal of which are represented in the subjoined sketches. Fig. 291 represents a simple and

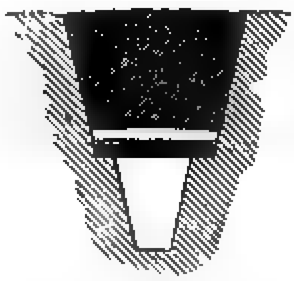


Fig. 291. Fig. 292. Fig. 293.
Drains.

economical system followed in countries where tubes or stones are expensive. It consists in forming shoulders upon the sides of the trenches, and laying upon them a thick sod with the grass downwards, the remainder of the trench being filled in with the materials thrown out from it, taking care to reject the denser and more impermeable earths. This description of drain is economically formed, but it does not last for any length of time, at least with sufficient efficacy.

Fig. 292 represents an economical form of drain for coun-

tries in which large quantities of water are to be removed, and where stone is cheap. The channel is formed by placing thin slabs on end, leaning against one another, and covering them with broken stones or gravel; the whole is then covered by sods and the lighter earths of the excavations, as before. If the waters draining through such channels do not contain any notable proportion of soluble salts, which they might gradually deposit around the broken stones, they will continue to flow for an indefinite period.

Fig. 293 represents the tile and shoe drains, which were much employed in England formerly, each tile being about 14 inches long, by 3 or 4 inches wide, and 4 or 5 inches high, and the shoes being of the same length, but a little wider than the tiles. Of late years, however, it has been the opinion of agriculturists, that perfectly cylindrical tubes are the most advantageous, not only on account of the greater facility of their manufacture, but also of the greater economy in their fixing. These cylindrical tubes are made of the same length as the earlier description of tiles, and of diameters varying from 1 to 3 or 4 inches.

When the soil is peaty, or a running sand, or when the nature of the materials through which the excavation is carried is such as to render it difficult to form and maintain the bottom of the trench in a perfectly straight line, the butting joints of the tubes will require to be protected by collars, which may be perforated with numerous small holes. Under ordinary circumstances, it will suffice either to use pipes with an end terminating thus , or merely with a straight end. In the last two cases, the trench should only be thrown out to the precise width necessary to receive the pipes; and in both it is absolutely necessary that the straightness and the uniformity of inclination of the bottom of the trench be rigorously observed.

Drains should not be made too long, because if the fall be

great there would be danger from the bursting of the pipes by the head of water; and the chances of choking are considerably increased, as well as the difficulty and expense of repairs. It is advisable to make the subdrains pour their water into a species of main of larger diameter, which subsequently should pour the collected stream into the general outfall. Mr. Parkes recommends that the submains should never much exceed 300 yards in length, and he usually makes the diameter of the lower half about $\frac{1}{6}$ greater than that of the upper, in order to insure the perfect discharge of the water. Under ordinary circumstances, however, it is preferable that the smaller drains should discharge into an open ditch, because the water would flow away more easily, and at the same time the repairs are performed with greater facility.

The length of the main drains may be greater, on account of their greater dimensions, but the condition above stated, of giving them an enlarged diameter at their lower extremity, must be observed. They are formed in the same manner as the subdrains, but, of course, in the lowest parts of the land; and it is advisable to place them at a slight distance below the subdrains, in order that these may discharge more freely. Their inclination must be greater, because the volume of water they have to transmit is also greater than that of the subdrains; and it is important to carry them at some distance from the hedges, or large trees, lest the roots should force their way into the pipes and choke them, because these are known to have a remarkable avidity for water, and are likely to force their way into the joints of the pipes. Lastly, it is important that the junction of the subdrains with the mains should not take place at right angles, but in an oblique direction, so as to avoid any interference with the velocities of the respective currents which might be likely to cause the deposition of any sand or mud in suspension of either of them. For the same reason it is advisable, that two drains coming

from different parts of the land should not be made to converge at the same point.

The distance apart of the drains will depend, in fact, upon their depth, and the degree of permeability of the soil; and this becomes one of the most important questions to be decided before commencing such works, for the greater the distance, evidently the less will be the number and the cost of the operation. Mr. Smith, of Deanstone, advocated the system of numerous drains, at comparatively shallow depths; whilst Mr. Parkes recommends that they be made deeper and at greater distances. The former made his drains from 6 to 8 yards apart, and about 3 feet deep; whilst the latter makes the distance from 13 to 20 yards, and the depth from 4 feet 6 inches to 8 feet. In fact, both parties may be in error in striving to enforce their respective system too rigorously, and a course of proceeding which may be eminently successful in one case may be very inadvisable in another. Thus, if a stratum of permeable materials exist, whose depth may be 6 feet, it is possible that a drain placed 5 feet below the surface may withdraw the waters from a distance of about 10 or 15 yards on either side. In such a case there would be a decided advantage in placing the drains at the greatest depths and distances, according to Mr. Parkes's plan. But if the soil itself be light, and at a depth of from 2 to 3 feet from the surface an impervious subsoil be found, it would be evidently absurd to carry the drains below the subsoil, because this would entirely destroy any lateral action of the drains beyond a distance of about 6 or 8 yards. In such cases, the system recommended by Mr. Smith is the more advisable; and, indeed, it happens in this particular branch of engineering, as in all others, that every individual case requires to be judged of and decided upon its own merits.

In Ireland the usual system latterly adopted appears to be so admirably suited to the class of materials most commonly met with, that an abstract of it is subjoined.

Minor drains are formed at distances apart varying from 21 to 40 feet; the depth is made 3 feet from the lowest point of the surface; the width from 15 to 18 inches at the top, and 4 inches at the bottom. These minor drains are parallel to one another, and only run from 150 to 200 yards without falling into either a ditch or a submain. In these drains a depth of 12 inches of broken stones, 2½ inches in diameter, is placed, care being taken that they be quite clean; a sod 3 inches thick is placed over them, and the earth is filled in. Sometimes pipes 2½ inches in diameter are inserted.

The submains are cut 42 inches deep, by 20 inches wide at the top and 12 inches wide at the bottom; they are carried along the low side of the field, about 10 feet from the fences, and are not allowed to run more than 300 yards without discharging into a covered or main drain. An open channel, 6 inches square, is formed, and above this the trench is covered and filled in as before with a thickness of about 8 inches of broken stones, carefully cleaned.

The open main drains are sunk to a depth of at least 5 feet; they are made 2 feet wide at the bottom, and the sides are thrown out to an inclination of 1 to 1, if the materials be such as to stand at that inclination, excepting in rocky countries, where the sides may be left at about ½ to 1. A minimum inclination of at least 4 feet per mile is required for these main drains. The dimensions of the covered main drains must necessarily depend upon the quantity of water they are intended to carry off; but generally speaking it is found to be sufficient to make them 1 foot square in the clear, with walls 6 inches thick, covered by flag-stones 3 inches thick, and filled in as before.

It appears that there is an advantage in executing the drainage of an agricultural district in dry weather, and in leaving the trenches open for a short time, in order that the ground may become warmer, and to a certain extent aërated, by being exposed to the atmosphere.

The measures to be adopted for the drainage of marsh lands must necessarily depend upon the causes which have superinduced this state. These causes are the following, at least in the majority of cases:—1stly, the superabundant humidity of the land may be owing to the fact that the subterranean waters are retained by beds of impermeable materials, and after saturating the lower strata, they are forced to make to themselves a vent upon the surface; 2ndly, it may be owing to the fact that the land is situated below the level of the surrounding country, and therefore receives the drainage from it; 3rdly, it may be owing to the existence of a river occupying a higher level than that of the marsh land itself.

Fig. 294.—Drainage.

The operations connected with the drainage of large marshes, fens, or bogs, require so serious an outlay that they can only be undertaken by large companies or by the State; but it frequently happens that small districts may be found in which a bed of clay occupies a position similar to that represented in the accompanying sketch, filling a depression upon the top of some permeable material, which last, in its turn, reposes upon a lower stratum of impermeable materials. In such cases the clay will prevent the water which soaks through the upper and exposed portions of the permeable stratum from flowing away at the lower point. The water will then accumulate until it rises to the level of the surface

of the clay, represented by the line A B, where it will overflow and form what are commonly called springs, which, unless provided with an outfall, will maintain the surface in a state of excessive humidity.

If, again, in the above sketch, Fig. 294, we suppose the basin-shaped depression shaded with interrupted lines to represent a bed of clay resting upon gravel, and to be filled in with ordinary soil, from the known impermeability of the clay it will retain all the water soaking through the soil to it, and in fact render the soil a complete morass, especially if the soil in question be surrounded by any eminences shedding their waters upon it.

In the illustration first supposed, the waters may be removed, either by bringing them to the surface at a point where a new and more effective outfall can be found, or by letting them escape to a lower level. In the first case surface drains are to be cut of a sufficient capacity to hold the waters likely to rise, and transverse outfall drains made to receive them. Borings should then be made in the surface drains, descending to the top of the upholding stratum, and the hydrostatic pressure of the supply, in such portions as are placed at a higher level, will cause the water to flow into the surface drains, until its level throughout the whole district will be found to be that of the drains. The outfall must be made as usual.

In the second illustration a boring, or borings, as may be required, are to be made through the impermeable stratum to the previous one upon which it reposes; or, in fact, a series of absorbing wells are to be formed, and the various surface drains made to converge to it. In the Treatise upon Well-boring and Sinking much information will be found connected with the principles of the action of such wells and their mode of construction. In these instances they will serve to carry the waters from the various surface drains into the lower strata, which almost invariably will be found to possess some

natural outlet, at a greater or less distance, in the shape of a spring.

Notwithstanding the progress of science in our times, Mr. Elkington's rules may still be quoted as being the simplest and most effective for the execution of the drainage of marsh lands formed by the outburst of land springs. They are as follows:—

1st. To find out the main spring or cause of the mischief.

2nd. To take the level of the spring, and ascertain its subterraneous bearings.

3rd. To use the augur to tap the spring, when the depth of the drain is not sufficient for that purpose.

It must be evident that if any district be situated so as to receive the waters flowing off from surrounding eminences, it will eventually be converted into a morass unless an outlet be provided. Should the district be small, this object may be effected, as before, by the formation of absorbing wells placed at the lowest points; but when its dimensions are considerable, the first operation to be performed will consist in forming a ditch all round the marsh, so as to intercept the waters flowing from the upper lands, and at such an elevation, and with such a fall, as to insure the discharge of any waters which may be poured into it either from above or from below. The banks, sides, and bottom of this ditch must be formed of impermeable materials. The ground contained within these banks must then be drained in the ordinary manner, and the drains made to converge to a point from which their waters may be withdrawn, either by means of an absorbing well, or by some mechanical contrivance, such as water-wheels, steam-engines, or windmills, setting in motion pumps, norias, or Archimedean screws.

If the marsh be owing to the existence of a river at a higher level, it must be treated in a similar manner to that just described, if the river itself cannot be diverted; or the river must be confined within impermeable banks, and the

waters draining from the low lands poured into it by some of the above-mentioned engines. It may, however, happen that the stream traversing the marsh may be subject to great and sudden floods; and in such cases it is necessary to form a double row of banks, of which the outer ones must be placed at a distance and superior elevation sufficient to carry off the increased volume of water flowing through them at such periods. The first banks then serve to contain the river in its normal state, the second will serve to contain it during floods; the intermediate bank, or zone, may be devoted to the cultivation of aquatic plants, such as osiers, willows, &c.; or it may be drained by a separate system from that of the marsh entirely protected.

Of the machines used to raise water in any of the supposed cases there are many varieties. Those hitherto applied may be stated to be—1, pumps; 2, Archimedean screws; 3, machines with buckets; 4, waterwheels with buckets, or what are called flash-wheels; 5, the water-pressure engines, hydraulic rams, rope pumps, &c.

Of these, the pump is the most effective when large bodies of water are to be raised from great depths, but it is exposed to the objection that the maintenance of the packing of the piston and of the pump barrel must be very expensive when the water to be raised is so much charged with earthy matter as must always be the case with that flowing from drains. If, therefore, the height to be overcome do not exceed 15 feet, it is usual to adopt other machines. Thus, in Holland the Archimedean screw is mostly used, when the height varies from 7 to 12 feet, and in the majority of cases motion is communicated by windmills; when the height varies from 3 feet 6 inches to 7 feet, however, flash-wheels are employed. In our own fen districts the scoop has been applied by Mr. W. Fairbairn with remarkable talent and success, in cases where the height to which the water had to be raised varied from 12 to 15 feet. In the East the noria (a machine consisting

of an endless chain bearing a series of buckets, dipping into the water at the lowest point of its course, and pouring it out as it passes the upper point) has been used from time immemorial. The fifth class of machines enumerated above are so seldom used for drainage purposes that it is not worth while to dwell upon them at present.

In Ireland, some large bogs have been drained upon the system adopted in reclaiming the bog of Allen, by withdrawing the water from below, and in this case it was attended with considerable success. The surface was firstly divided into fields of an oblong figure, and of about 5 or 6 acres area, by open drains. Augur holes were driven at distances of about 33 feet down to the rock, and at a level of at least 1 foot above the surface of the water in the drain. Curved pipe tiles, 1½ inch diameter, were inserted into the holes, so as to throw the water into the centre of the drain. These drains were made about 6 feet deep. On the Chat Moss drainage no effort was made to withdraw the deeper-seated waters, but all the measures adopted were designed merely with reference to those flowing upon the surface. Square enclosures were formed, 100 yards long by 50 wide, by means of large open drains, 3 feet 9 inches deep at the minimum, 8 feet wide at the top, and 1 foot 8 inches at the bottom. Covered cross drains were formed communicating with the open ones, and with a width of between 12 and 14 inches as far as the shoulder, placed about 2 feet 2 inches from the surface; below which point they were carried to a further depth of about 16 inches, with a width of 8 inches: these cross drains were placed at distances of about 6 yards from centre to centre. No tiles or pipes were used, the bottom of the drain filling being formed by the surface spit raised from the moss.

[In the reign of Charles I. it was determined to drain the great level of the fens—an extensive district of low marshy

land on the east coast of England, bordering upon the river Humber, the Witham, the Ancholme, the Welland, the Nene, and the Ouse, called the Bedford Level, from the name of the Earls of Bedford. The fen districts present the largest work of the kind in the world. They appear to have formed, at one time, an estuary of the Wash, into which the rivers above-named, and also the Glen River, were discharged. In the north of Lincolnshire, the fens do not extend more than four or five miles inland. In the south of that county they are 20 miles in width, and between Lynn and Peterborough, which is the widest part, they are 30 miles wide. Equal width prevails for some miles further south, until the fens are arrested by the oolitic elevations of Huntingdonshire and Cambridgeshire. They terminate at a few miles south of Ely, making a total length of fen district of about 130 miles.

The extent of country drained by the Wash, includes the entire counties of Cambridge, Huntingdon, Bedford, Northampton, and Rutland; nearly one-half of Norfolk, one-third of Suffolk, one-half of Buckinghamshire, three-fourths of Lincolnshire, and a small part of Leicestershire. The area comprised altogether amounts to about 5,000 square miles.

The Bedford Level is divided into three parts, called, respectively, the South Level, the Middle Level, and the North Level. The South Level is drained by the Ouse and the Bedford Rivers, which have been variously treated. The North Level was, for a great number of years, drained by the Wisbeach outfall and the crooked course of the Old Nene River. The Middle Level presents a much more complicated system of drainage than the other levels. The principal artery of this area is the Old Nene River.

The rivers already named form the present water-drains of the fen districts. As fen rivers, the quantities of fresh water transported to the sea by the Witham and the Welland are inconsiderable. The Ouse and the Nene are the more important drains. They rise in the same county, and, after

following a direction nearly at right angles, discharge themselves into the Wash within a few miles of each other. The Ouse is probably one of the most tortuous rivers in the county. Its main branch rises at Gentworth, about 10 miles north-west of Buckingham, which is about 80 miles from Lynn, though it traverses about 160 miles in its course. The Nene originates in two springs north and south of Daventry. Its course is easterly to Northampton, where it becomes navigable. The direct distance to the outfall at the sea is 60 miles, but the course of the river is nearly 100 miles in length. At Peterborough, it enters the fens. It is chiefly conducted through this region by artificial cuts, so that its original channels are, in some places, hardly traceable. The valley of the Nene is lost 30 miles above its outfall. The valley of the Ouse does not extend further than St. Ives, and it runs a course of 50 miles afterwards between this town and Lynn. The valley of the Welland and that of the Glen—a secondary river which runs into the Welland—terminate on the borders of Lincolnshire. It is apparent that all the rivers just noticed seek a common outlet, over lands which are no higher than the beds of the rivers; and that they are only prevented by embankments from overrunning the soil.

The plans of Cornelius Vermuyden, a Dutch Engineer, for draining the fens were executed in the middle of the seventeenth century, and they were successful, to a certain extent, in draining the level. A sluice was placed across the Ouse at Denver, about 15 miles from the sea at Lynn, where the Ouse enters the Great Wash, so as to exclude the tidal waters, leaving the channel of the Ouse above that sluice for discharging the fresh waters only. These it was proposed to conduct from all parts of the land by small lateral drains or canals, carried to the river in courses as direct as was practicable, having sluices at their junction with the river to prevent the floods from entering them and

covering the adjacent lands. A new channel, also, about 20 miles long, called the Bedford or Hundred Foot River, was cut, for a part of the river Ouse, from the point where Denver Sluice was erected to the old channel of the Ouse at Earith, where another stanch or sluice was placed for preventing the tide from going beyond that point.

Vermuyden considered that, by adopting this plan, and having only the fresh water to contend with, he would get rid of that powerful enemy to drainage, the tide, and would have only to deal with the fresh water. This plan, for a time, answered tolerably well, and a considerable improvement in the drainage was effected. But the mouth of the channel of the river Ouse, which is the chief outfall for the drainage of the district where the Bedford Level is situated, being deprived of its accustomed and natural scouring power of tidal water, became so obstructed by shoals that the land water could not pass off to the sea. In proportion as the drainage became defective in process of time, as it necessarily did, windmills were erected to work scoop-wheels, with a lift of 4 or 5 feet, for raising the water out of the lateral canals into the river. In 1713, Denver sluice was undermined and blown up by the floods, and the tide recovered to some extent its ancient receptacles; but the sluice was rebuilt after a few years on the old system, and the drainage and the navigation became deteriorated as before. The principal defect existed immediately above the town of Lynn, where the river took an extraordinary bend, almost at right angles to its general course, for a length of 5½ miles, forming almost a semicircle of a diameter not exceeding 2¾ miles. In addition to this diversion, by which the fall or inclination of the current was lost, the channel was so irregular and disproportionate in width, and so much encumbered with drifting sands, that the tidal and fresh waters were unable to force their way through them. Thus the drainage waters were penned up above, and, being detained, they formed a tranquil

pool, which during floods frequently broke the banks and inundated the surrounding country; and the channel, moreover, being deprived of its natural scour, silted up in the same proportion. The remedy for this great evil, first proposed in 1724 by Bridgeman, and approved by many other engineers, was to cut off the bend in the Ouse by opening a comparatively straight channel between the two extremities. The Eau Brink Cut, executed for this object, was opened in 1821, and very beneficial effects immediately followed. The extraordinary wet winter of 1821, which succeeded, proved its success beyond doubt. Soon after the cut was opened, the low-water line in the Ouse immediately above the cut fell 5 feet. The additional fall augmented the inclination in the current, which acquired increased velocity and greater power to scour away and remove the obstacles in the bed of the river, and to cause the discharge of a greater quantity of water in the same time, as well as a longer period for discharging it. In consequence, it became practicable to lower the sills of the Denver sluice 6 feet. The country was greatly benefited by the improved discharge and the better drainage. The tidal waters, moreover, being freed from the shifting sands and circuitous course of the old channel, and being confined as one mass in the new direct channel, acted with greater effect. Finding their way upwards, and becoming united with the fresh waters, and enlarging and deepening the channel above, the channel was kept open to its proper dimensions. Consequently, both the drainage and the navigation were improved by the Eau Brink Cut, whilst an area of 300,000 acres, drained by the Ouse, was brought into profitable cultivation. The improvement was carried still further in enlarging the cut by one-third, making a total augmented fall of 7½ feet in the current at the upper end.

The drainage, nevertheless, was not quite satisfactory, and some further works were undertaken in 1850—steam power for lifting water in the neighbourhood of Whittlesea Mere,

and an improvement in the outfall below Lynn—under the direction of Sir John Rennie and Mr. Robert Stephenson. The "New Cut" was constructed—being virtually an extension of the Eau Brink Cut—a straight cut 2 miles in length, and from 600 to 700 feet wide: discharging the waters into the Norfolk estuary at a lower point, and so improving the tidal action as to increase the gain at Lynn, which had amounted to 7½ feet in consequence of the Eau Brink Cut, by 6 feet additional, making a total augmentation of 13½ feet in depth.*

A similar operation was performed by Telford and Rennie on the river Nene, in 1829, at the Nene outfall, which commences about 5 miles below Wisbeach and terminates at Skates Corner; making a length of nearly 5 miles, where it joins the estuary of the Wash. The beneficial effects of this work were decisive. The low-water mark was lowered 10½ feet, and a district of above 100,000 acres in extent was completely drained and brought into cultivation, which formerly was, for most part of the year, a stagnant marsh. The tide rises 14 feet at Wisbeach, and vessels of 200 tons come up to the town, whereas previously the river was only navigable for small sloops.

An extensive plan for the interior drainage was next designed and executed by Telford in 1830. It consisted of one main drain, with two subsidiary smaller drains, extending above 20 miles, to Thorney, to bring down and discharge all the waters from the low fen-land districts into the upper end of the new outfall, by means of a capacious new sluice with self-acting gates, by which the water from the drains is discharged into the Nene, so long as the level of the water in the drains is higher than that of the river. When the water in the river is higher, the sluice-gates close, and prevent the water of the river from entering the drains. This plan of

* There is uncertainty as to the amount of the extra depth effected by the Eau Brink Cut.

Telford's resembled one previously proposed, on a more extensive scale, by Rennie, for the same object, and accompanied by the important addition of catch-water drains.

Turning to the district of fen-land drained by the river Witham, Holland Fen, lying to the south of the Witham and south-westwardly of Boston, was treated for drainage as early as 1638, when a sluice known as the "Grand Sluice," now the "Black Sluice," was built abutting on the river a little below the town of Boston, and a drain 8 miles long, reaching to Swineshead, was cut to bring the waters of the fen to this outfall. Opposed by the population, the adventurers renounced the scheme, which had been partly worked out, the works fell into disrepair and decay, and the land relapsed into its primitive condition—a vast and desolate fen.

In 1767, another and a successful attempt was made to drain Holland Fen. The whole of the waters were brought to the outfall at Black Sluice by a drain, known as the "South Forty Foot," running through the middle of the fen, for the most part at right angles to the Witham, upwards of 20 miles in length, wide enough and deep enough to allow of the passage of boats for the conveyance of produce. The area of land drained by this cut amounted to upwards of 64,000 acres. In addition, the drainage of 30,000 acres of land that had been drained by the river Glen was directed to this new cut; and, in fact, the waters of the Glen itself, previously discharged into the river Welland, were directed towards the South Forty Foot. In the year 1846, the drainage became defective, partially owing to the subsidence of the land occasioned by the drainage of water from it. The drain was, in consequence, lowered from 4 feet to 5 feet throughout, and a new outfall sluice was made, having three openings of 20 feet each, one of which was constructed as a lock for navigation. The sill of the new sluice was 6 feet below the level of that of the old sluice. The fall of the drain

is 3 inches to a mile. The whole of the fen is under cultivation.

The district to the south of the Witham, between Holland Fen and the river, is drained by the " North Forty Foot"—a drain which was cut about the year 1720—running parallel to the Witham. It discharges the waters at the same outfall, the Black Sluice, as that of the South Forty Foot.

The eastern portion of the fens, lying to the north-east of the Witham, called the fourth district—the East Fen, the West Fen, and the Wildmore Fen—are the lowest in level of the fens in the district. From an early period the westerly portions of these levels discharged their waters into the Witham, at a point about two miles above Boston, through a sluice called Anton's Gout, and the remainder was drained through Maud Foster's Gout, a sluice discharging the waters into Boston Haven, a mile below the town. In 1801, an Act was obtained for the drainage of these fens, the area of which was computed at 40,000 acres, although at present there are upwards of 62,000 acres taxable by the Drainage Acts, and there is a watershed of upwards of 82,000 acres. Mr. Rennie was consulted, and, in 1806, he proposed and executed a complete system of drainage. Perceiving the inefficiency of the river as a means of drainage, he proposed that it should be improved by straightening its course, and increasing the capacity of its channel. But he was forced by opposition to carry his main drains into the river below Boston. He divided the drains into two classes. One class was called " catch-water drains," which, running along the base of the hills surrounding the low lands, intercepted all the waters of the high lands. These waters were conducted by the catch-water drains into a main drain—the old Maud Foster Drain, which was enlarged—from which the waters were discharged by a self-acting sluice into the Witham, below Boston. The low-land waters, thus freed from the high-land waters, were conducted by separate drains into another main drain at

Hobhole, about three miles lower down the Witham, where there was more fall. By this means both classes of waters were discharged independently of each other. But provision was made for discharging, if necessary, all the water by the lower drain at Hobhole. This combined discharge did become necessary, and the district, formerly a stagnant marsh, was converted into corn-fields.

The theory of the "catch-water" drain system is obvious. Previously, the waters flowing from the higher levels found their way quickly off the lands and filled the drains, damming back the water in the ditches on the low lands, where it remained until the upland waters had found their way to the sea. The catch-water drains, skirting the whole district bounding the fens, prevented the high-land waters from entering the low-land drains; and so the low-land waters were allowed to flow off without interruption.

The Witham was originally a tidal river. By neglect it became much silted up; and, as a remedy, a sluice was erected at Boston, about the year 1530, under the advice of a Dutch engineer, May Hake. The sluice failed to perform what was expected of it. In 1751, Boston Haven, then a reach of the river below Boston, partially silted up, so that, though large vessels of 200 tons formerly came to Boston, only sloops of 50 or 60 tons could arrive there. The North Forty Foot was blamed for abstracting the water supply of the river. A new sluice, called the Grand Sluice, was substituted for Hake's sluice, and was built across the river a short distance above the town. It consisted of four arches, about 21 feet wide, one of which was used as a lock. Self-acting sea-doors were hung on the outer side of the sluice; and, in the interior, slackers or draw-doors were hung, for regulating the level of the water in the river. The cost of the works amounted to £60,000. It was found, after all, that the drainage and the navigation were hostile and incompatible with each other. The river, left to itself, silted up.

The land in the East Fen, originally the lowest in level, became lowered by the thorough drainage to the extent of from 1 to 2 feet—in some cases 2½ feet—below its original level. The upper surface of the soil, full of peat and organic matter, was saturated with water; and as the soil has, in the course of years, been worked and cultivated, the organic matter has gradually decayed. By, drainage, the soil has become less spongy and more compressed, and therefore it subsided from its original level. Hobhole Drain, in consequence, did not work so effectually as at first; and water, in wet seasons, was left for a considerable length of time on the lowest lands. Sir John Hawkshaw was, in 1861, consulted on the question. He proposed alternative remedies. One was, by an improvement of the Haven, to lower the low-water mark; the other was to erect a sluice about the middle of the drain, and a steam pumping engine to lift the water from the low-level portion of the drain at the lowest lands—that is to say, at the upper section of the drain—to the level of the higher level portion. As the first alternative involved the co-operation of all the trusts interested, the second was adopted, and pumping machinery was, in 1870, erected at Lade Bank, a station on the upper part of Hobhole Drain.

The works comprised—1st, a sluice or dam having three water-ways, each 12 feet wide, and as high as the dam, closed by ordinary swing-gates, in pairs. The central water-way is also used as a navigation lock, having two pairs of gates. 2nd. Two pump-wells, each 12 feet wide, also closed by ordinary swing-gates, in pairs. The engine-house is built over the wells. 3rd. The boiler-house, with storage for coals, and the chimney adjoining. In each pump-well there is a double-suction Appold centrifugal pump, having a horizontal fan 7 feet in diameter and 2 feet 2 inches deep, on an upright shaft. On this shaft, at the upper end, a bevel pinion is keyed, which is driven by a mortice bevil wheel on the crank-shaft of the steam-engine. There is a pair of

engines, self-contained, on one bed-plate, to each pump. The engines are condensing, vertical, and direct-acting, with a rocking-beam parallel motion. The cylinders are 30 inches in diameter, with a stroke of 30 inches. They are worked expansively, the steam being cut off at one-fourth of the stroke; and the engines make 36 revolutions per minute. The two pairs of engines are supplied with steam from six Lancashire boilers, 6½ feet in diameter and 23 feet long, with two tubes 2½ feet in diameter. The pressure in the boiler is 45 lbs. per square inch.

The excavations, buildings, and machinery were executed in one contract by Messrs. Eastons, Amos, and Anderson, for the sum of £17,000.

The entire drainage of 35,000 acres is now performed by means of this machinery. It was well tested during the winter following the time of its completion. Within 36 hours, on the 7th and 8th of December, 1870, there was a fall of rain equivalent to 0·94 inch, whilst the total rainfall of that month was fifty per cent. in excess of any winter month during the previous 40 years. It appeared that, whereas in January, 1867, with a rainfall of 3·32 inches, an area computed at from 10,000 to 12,000 acres was placed under water for several weeks, the whole of the district was kept perfectly clear of water in the month of December, 1870, with 5·28 inches of rainfall, by means of the new machinery.

The total working expenses for the year ending March 31, 1872, amounted to £573. For the preceding year it was £490, or from 3½d. to 4d. per acre drained. The cost of the works was at the rate of 10s. per acre, which, taking interest at 4½ per cent., represented 5d. per acre.

STATISTICS OF WORK.

	Year ending 31st March. 1871.	1872.
Average number of turns of engines per minute	36·02	38·2
Average lift in inches	44·77	45
Sum of hours worked by both pumps . .	794·25	980·5

Weight of water discharged, tons . .	13,564,190	18,296,130
Coals consumed during working hours, tons	328	397·25
Engine oil consumed, gallons . . .	25·75	20·25
Tallow consumed, lbs.	181	135
Waste used, lbs.	135	85
Wages paid:—		
First and second drivers, yearly . .	£158 12s.	£158 12s.
Boy, yearly	£15 12s.	£18 4s.
Firemen, 2,085½ hours at 3½d. per hour	£30 8s.	£29 13s.
Ditto 2,033 " "		

The Middle Level presents a complicated system of drainage—a network of leams, drains, eaus, and rivers, running in all directions: in some instances, at right angles to the direction of the outfall, in others in a direction nearly contrary. The Old River Nene—the principal artery—takes a very circuitous course. It runs through Whittlesea, Ugg, and Ramsey Meres, and thence to March and Upwell, where it is connected with the Ouse by a junction with Popham's Eau and Well Creek. It had, for many years past, been a source of contention between those using it as a navigation, and the owners of land requiring to make it available for drainage.

In 1842, Mr. James Walker reported on the drainage of the Middle Level, affecting an area of 140,000 acres, exclusive of the high lands about Whittlesea and March. With the exception of these lands, the whole of this vast area was drained by artificial means. He proposed a main line of drain to commence at the upper end of the Eau Brink Cut, above the Marshland Sluice, to Caldecot Farm, on the west side of Whittlesea Mere. It was to be level for its whole length, which was 31 miles; but it was only partially constructed. It extends 11¼ miles from the point of junction with the Eau Brink Cut, and communicates with Popham's Eau and the Sixteen Feet River, which are two of the main drains of the Middle Level. A sluice, known as the St. Germains' Sluice, was built at the outfall or confluence of the main drain with the Ouse.

The main drain, or, as it is called, Walker's Cut, as at first made, in 1847, was 50 feet wide at its lower end, 40 feet wide at Well Creek, and 30 feet wide above Well Creek. The level of the bottom, at the outlet, was 5 feet below low water in the Ouse, and that of the sill was 8 feet below the same level. It had an inclination upwards of 1 inch per mile to Well Creek, where sluices were placed to keep up the level of the water in this river for navigation. The sills of the Well Creek sluices were laid at the same level as that of the outlet sluice at the upper end of Eau Brink Cut, and they consisted of three openings of 20 feet each. In 1848, it was determined to deepen, with a few exceptions, all the main rivers and drains of the Middle Level—about 110 miles—from 4 to 6 feet, with widths at the bottom of from 12 feet to 30 feet; to make new cuts or junctions; and to construct locks at Upwell, Horsway, and Ashline, for navigation. In consequence of these works, which were all completed in 1852, the water in the rivers and drains was lowered 6 feet.

In 1857, Mr. Walker's original design was further carried out by the deepening of Walker's Cut 4 feet, or to 1 foot below the sill at the outlet, with a level bottom for its whole length of 11¼ miles. The width at the bottom, thus deepened, was 48 feet. The slopes of the sides are 2 to 1; and, according to Sir John Hawkshaw, writing in 1863, the level of the bottom is 7 feet under low water of spring tides in the river Ouse, at the point where the drain enters the river. The rise of the tide in the Ouse at that point is about 19 feet at spring tides, and the level of the sill of the St. Germains' Sluice is 6 feet below low water of spring tides. The bed of the drain, at this place, is of soft blue clay, and the sides consist of variable thicknesses of soft blue clay, peat, yellow clay, and surface soil. Of these materials the side banks above the level of the ground are composed.

On the 4th of May, 1862, the St. Germains' Sluice gave

way. The tidal waters were admitted from the river into the drain, and rushed up, and again poured out of the drain with great velocity. For a distance of 20 miles the waters ebbed and flowed. In the course of a few days the banks were breached at a point about 4 miles above the sluice, and an area of upwards of 9 square miles, or about 6,000 acres of land, were inundated.

Sir John Hawkshaw, who was intrusted with the work of repair, determined to construct a permanent coffer-dam of pile-work across the drain, as the only structure that was likely to answer the purpose. Two rows of sheet-piling, 25 feet apart, were driven across the slopes at the sides of the drain, and the intermediate portion, across the bottom and the lower parts of the sides, consisted of pairs of whole timbers driven down, 7½ feet apart from centre to centre; and the intervening spaces were occupied by panels 7 inches in thickness. The dam was strongly fortified by struts and ties.

Sixteen siphon pipes of cast iron, 3½ feet in diameter, were laid across the dam, for discharging the waters, at an inclination of 2 to 1, at each side. The upper portions of the siphons are horizontal, at the crown of the dam; and the ends are also horizontal at the bottom of the drain, being laid 18 inches below the level of low water of spring tides, measured to the upper side of the pipes. These are, therefore, always under water. The top of the siphon is 20 feet above the same level. The siphons are set in action by exhausting the air from the inside, by means of an air-pump worked by a steam-engine. The air-pump has three 15-inch cylinders worked together by means of a three-throw crank-shaft, with a stroke of 18 inches. The engine is of 10 horse-power, having a 12-inch cylinder with a stroke of 20 inches.

The works for the drainage of the Ancholme Level, consisting of about 50,000 acres, extending 24 miles south of the Humber, were executed to the designs of Mr. Rennie

and Sir John Rennie. The river Ancholme takes its rise a little to the north of Lincoln; and, after a course of about 85 miles, passing through the centre of the district, it discharges itself into the Humber about a mile to the west of the village of Ferraby. The valley varies from one mile to three miles in width. At a place called Bishop's Bridge, about 20 miles from the Humber, and at the southern extremity of the level the Ancholme is joined by a large brook, the Rasen, which brings down considerably more water than the Ancholme. One day's flood would cover the whole level to a depth of 2¼ inches. The following statement affords an idea of the quantities of water that were to be dealt with:—

	Cubic feet.
The streams on the east side produce . . .	48,014,000
Those on the west side	24,500,000
The Ancholme and the Rasen	36,000,000
Sundry small streams	32,000,000
Total in one day's flood . . .	140,514,000

Whilst the area of the valley amounts to 50,000 acres, it receives the water from 150,000 acres of high lands bordering on the east and the west sides.

Sir John Rennie recommended that the plans of the catchwater drains, proposed by Mr. Rennie, should be carried out to their full extent; that the main river, the Ancholme, should be straightened, widened, deepened, and enlarged to double its capacity; that a new sluice should be constructed at Ferraby, with its sill laid 6 feet lower than the old sill, together with a new lock 20 feet wide, so as to serve the double purpose of accommodating larger vessels and of acting as an additional discharge for the drainage waters during periods of flood; that all the old bridges should be removed, as, during floods, they kept back the waters, and were serious obstructions to the drainage; that a new lock should be constructed at Haarlem Hill, 18 miles above Ferraby Sluice.

That as, during floods, the Ancholme and the Rasen brought down a considerable quantity of sand from the adjacent hills, so as sometimes to block up the main river and the drains, and thus to prevent them from discharging their waters, and thus causing inundation of the adjacent lands, it was further proposed to construct a large overfall and weir, with an extensive reservoir on the lower side, to catch all the sand and mud which was brought down from the upper part of the country, and thus to prevent its falling into the main river. From the reservoir, the mud and sand would be occasionally removed. It was also subsequently recommended that there should be similar overfalls, weirs, and reservoirs at all the minor streams and brooks where they united with the level.

The works of the Ancholme drainage were executed in conformity with the terms of Sir John Rennie's report, and were completed in 1844. They were attended with all the success which was anticipated. Sir John Rennie briefly indicates the leading principles of drainage thus practically established :—

1. The formation of catch-water drains, which separated the high-land waters from the low-land waters, and conveyed each class of waters to independent sluices at the lowest practicable outfalls. This system was first practised by Mr. Rennie, about the year 1801, in the Witham drainage.

2. The straightening, deepening, and general improvement of the main river, separating as much as possible the navigation from the drainage.

3. The formation of overfalls, weirs, and reservoirs, for arresting the sand and mud, and preventing the drains from being choked.]

CHAPTER X.

DRAINAGE OF TOWNS.

THE DRAINAGE OF TOWNS is a subject of such manifest interest to the community at large, that the discussion of the best and most efficient system to be adopted has occupied the attention of legislators and engineers at all times. There are two branches of the subject which may be considered to be sufficiently distinguishable from one another for the purposes of classification, and which may be, and often are, treated in practice upon very different principles. These subdivisions are—1st, the consideration of the means for removing surface or drainage waters; and, 2nd, of the consideration of the means of removing all excrementitious matters in such a way as to insure their most effectual removal without annoyance, and their economical adaptation wherever possible.

Wherever a large and highly civilised community assembles it becomes frequently difficult to separate the two classes of matters to be removed, especially as existing municipal arrangements complicate the question in an infinite number of ways. Cities grow, without much apparent reason for the particular manner in which the increase affects their plan; very rarely, indeed, is it possible to predicate, and to provide for, the eventual wants of their population, not only because the distribution of cities may alter, but also because from time to time changes are effected, even in national habits, which defy all previous calculation. Thence it is that we find both the want of systematic arrangement in our own country,

and the excess of it in France, equally sources of difficulty in the adaptation of modern refinements. But we have at least this advantage—that having done little, we have less to undo; and, after all, it appears to be the wisest course to deal with these questions as they arise, without endeavouring to restrain the freedom of action of those who are to succeed us.

However, in all modern cities the tendency certainly is to divert house sewage into the public drains, especially in our own country. There are still many towns in which the old system of drainage for surface waters, and cesspools for house refuse, prevails; but, compulsorily, they are diminishing in number every year. There are some conditions which render it doubtful whether the concentration of the two systems in the same discharging drains be desirable, at least under all circumstances; and in this, as in all other branches of engineering, no inflexible rule can be said to exist. Owing to the nature of the soil upon which a town is built, its configuration, the character of the outfall, and of the country round that outfall, a course highly advisable in one case might be objectionable in another. These modifying causes will be examined successively; stating before so examining them the general conditions to be fulfilled. Some of these considerations will be found to apply to the discussion of the questions connected with the water supply of towns,

The conditions required to be fulfilled are, as before stated, that the whole of the surface and land waters be removed, and that the house refuse be carried away effectually and inoffensively. The latter will depend, in quantity, upon the population, and the greater or less abundance with which water is supplied for domestic use. In England, it is only in exceptional cases that the average number of inhabitants per house exceeds 6; whilst in France and some parts of Scotland it may be occasionally as many as 40. Upon a copious distribution of water being effected, it is usual to

calculate that every individual would give rise to a consumption of about 20 gallons per day; and probably of this total quantity about 16 gallons may find their way into the sewers from the various dependencies of houses. Sewers, then, if designed to remove all waters, must be of sufficient capacity to discharge a volume calculated upon the above supposition, together with any storm-waters which may fall. It has been observed by Mr. Phillips, that the greatest flow of house sewage takes place between the hours of 11 and 1; and that in each of those hours at least ⅛th of the total daily discharge finds its way into the sewers. The capacity of the latter must, then, be made such as to discharge the greatest quantity of storm-water falling in one hour, supposing it so to fall when the house drainage also furnishes the greatest volume.

The soil upon which a town is built may influence the character to be given to its drainage, either as it may favour or impede the transmission of what are called land springs. Thus in many parts of London, and also in the town of Southampton, there exist small elevations, the surface of which consists of an impermeable brick earth, lying upon a stratum of gravel and sand, this last again capping the stiff blue clay known as the London clay. In many other cases the upper stratum of brick earth is wanting, and the gravel forms the immediate surface stratum; whilst in others, again, both are wanting, and the London clay is entirely exposed. The drains and sewers to be laid in between the points B and

Fig. 295.—Influence of Soil on Drainage.

C, of such a formation as is represented, need not be made of a greater capacity than is required to remove the surface or the house waters supplied by the district; but those to be

laid between C and D, and still more those between D and E, must be able to receive the waters filtering through the bed of gravel. Near London, the exposed surface of gravel is generally so small that the water yielded by it does not require to be taken into account; because the dimensions given to the sewers to enable them to carry off storm-waters are more than sufficient to relieve the strata traversed by these springs, which are necessarily characterized by a certain degree of regularity in their flow. At Southampton, however, the extent of superficial gravel is, proportionally, infinitely greater; and it is found that, after a continuance of wet weather, the whole of the lower portions of the gravel become charged with water to such an extent as to inundate all the basements below the level of the natural ground, unless where large sewers are formed, so as to intercept the flow of the subterranean waters.

In some parts of Paris the same phenomena occur upon a larger scale and with greater regularity than in the cases above cited. There a considerable portion of the city is built upon what formerly constituted a marshy plain, between the river and the hills of Belleville and Montmartre. The lowlands are situated upon a calcareous formation, called geologically "the lower fresh-water limestone," which allows the water to infiltrate with great difficulty; and the several hill-sides are successively formed of the gypseous deposits, with their associate marls, capped by a deep stratum of sand and sandstone, occasionally covered by the upper fresh-water limestone. In the direction towards Belleville the sands occupy a considerable breadth of country, and receive a copious supply of water during the rainy seasons. At the same time the various hills present steep escarpments, so that the storm-waters, falling upon them, flow away with great rapidity. It follows, from these combined circumstances, that in order to obviate any inconvenience from these respective sources, the intercepting culvert, executed along the line of greatest depression,

has been formed of much larger dimensions than the area it immediately drains would appear to require, without, however, preventing the occasional flooding of the lower parts of this quarter of Paris.

If the geological structure of the soil of a town in some cases appear thus to increase the difficulties connected with its sewerage, there may be others in which it produces precisely opposite results, so far at least as the removal of surface waters are concerned. Thus, in Weymouth, the portion of the town constituting the ancient borough of Melcombe Regis, is constructed upon what is, in fact, the shingle bar thrown across the mouth of the Wey. All that is required, then, to remove the surface waters is to form openings through any paved roads or courts—absorbing wells, in fact —and the waters immediately sink to the level of the sea. In some parts of Liverpool, also, advantage is taken of the absorbent nature of the gravel to allow the surface waters to soak into it; for the drains are occasionally executed in the lower portion of bricks laid dry, whilst they are only set in mortar in the upper portion.

The configuration of a district, meaning by that term the general conditions of its division into subordinate districts of hill and dale, will also influence the system of sewerage to be adopted, insomuch as it may affect the number, dimension, and direction of the main sewers. New and distinct outfalls may be required for the several portions, and frequently, according to the final mode of disposing of the sewage, distinct establishments may be required for its preparation.

The influence of the outfall is very great, for it may easily be conceived, that if a system of sewerage be made to discharge into a watercourse flowing always in one direction, as in the case of all cities situated upon rivers above the tidal range, provided the outlet be so situated as to insure a constant flow, no necessity can exist for providing against an accumulation of the sewage waters. But in tidal rivers, it

frequently will occur that the mouths of the sewers will be blocked up by the rise of the tide for intervals, varying of course with the peculiar laws of the tides of the precise locality, and with the levels of the mouths. It becomes necessary in such cases to construct the lower ends of sewers so discharging of sufficient size to admit of their containing the waters at any time likely to flow into them during the intervals of their suspended discharge; and also to make them of sufficient strength to resist the hydrostatic pressure of any accumulation in their more elevated portions. It is to be observed, that the above reasoning only applies in those instances where the sewage is poured into the rivers directly, without being in any manner usefully applied, either in the arts or in agriculture.

The quantity of storm-waters flowing from any given district within a given time, has frequently been alluded to as one of the most important elements in the determination of the size of the sewers. It is naturally very variable, not only according to the latitude of the localities considered, but also according to particular seasons of the year: it depends, in fact, upon the frequency and the violence of sudden atmospheric changes, rather than upon the average state of the weather; and, as the sewers must be constructed so as to carry off the maximum rain-fall, the ascertaining accurately what the latter may be is an indispensable condition for the determination of their capacity.

The average distribution of rain in different localities has already been treated in a summary manner. It may suffice, then, at present, to observe that torrential rains occur with the greatest frequency in countries near the tropics, but that higher latitudes are by no means exempt from them. At Rome, where the average annual fall is about 2 feet 8 inches, showers have been observed of 17 hours' duration, with a total fall of not less than 5 inches. At Marseilles, in a shower of 14 hours' duration, 13 inches

of rain have fallen; and at Arles, in 12 hours, nearly 8 inches. At Southampton, the greatest fall which has been noticed is about 2 inches in 10 hours; whilst in London as much as 6 inches of rain have fallen in 1½ hour. The latter observation would appear to have been influenced by some very exceptional phenomenon, perhaps of the character of a waterspout; but it appears, from numerous other observations in England, that storms of a similar nature to that mentioned as observed at Southampton, are of sufficiently frequent occurrence to justify the assertion of the rule, that "when sewers are constructed to carry off storm-waters, they should be of a capacity to discharge a proportion of a 4-inch rain fall in 24 hours, varying according to the character of the district."

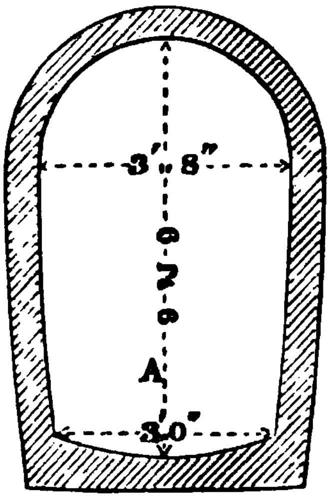
The proportion so flowing into the sewers will depend upon whether the district be rural or urban; and, in the latter case, upon its configuration and the degree of permeability of the soil. It is usually calculated, that in the open country about ⅓rd of the rain-fall finds its way directly into the natural watercourses; in ordinary country towns about ⅔rds are estimated to flow at once into the sewers; and, perhaps, in large densely-populated towns, it would be safer to calculate upon ¾ths of that quantity as likely to reach them.

If a demand existed for the application of sewage manure to agricultural purposes, and if a sufficiently copious household supply existed to insure the flushing of the drains, it would unquestionably be preferable to keep the two classes of sewage distinct, because the casual introduction of large quantities of storm water must superinduce an irregularity in the quality of the sewage, which could not fail to be prejudicial to its application. Unfortunately, there appear to be difficulties attending this practical application, at least in the present state of agricultural and engineering science; and, notwithstanding the bold assertions of some modern

authorities, who from their position might fairly have been expected to exercise greater reserve and discretion, all the attempts to apply liquid sewage manure have been hitherto most signal economical failures. The separation of the two classes would produce in many cases an additional advantage, from the fact of the smaller sectional area required for the sewers, and, consequently, from the increased fall, or, where that could not be obtained, from the greater elevation at which the outlet might be established.

The form to be given to sewers may sometimes require to be different from what it is at others, owing to the necessity which may exist to visit and cleanse those which have not either a sufficient fall or a sufficiently copious supply of water to keep themselves clear. The only invariable rule to be laid down upon this subject is, that "the wet contour should be made to bear the smallest possible proportion to the sectional area," for the simple reason that the friction is always in the direct ratio of the surface upon which it acts. It follows that, wherever it is possible so to execute them, sewers should be made of a circular section. For house sewers there can be no difficulty on this score, because the introduction of the tubular drains has furnished not only the most efficient, but the most economical, means of execution. The only remark which appears to be required on this subject appears to be that, at the present day, the tendency is to execute them too small; and that there is danger of their choking if used of less than 4 inches in diameter. For secondary main drains the same system of tubes may be applied to a certain extent; but when the length becomes considerable, there will be found so many probabilities of obstruction, and so great danger from the accumulation of gas evolved from the water, that it is very questionable whether tubes should ever be used without the formation of side entrances for their examination and repair at maximum distances of $\frac{1}{8}$th of a mile, or without frequent opportunities of com-

munication with the atmosphere. Contrary to the fashionable theory, it may perhaps be more advisable to construct a main drain, intended to receive several secondary mains of sufficiently large sectional area to allow of its being visited and repaired without entailing the necessity for opening the ground. The form of main sewers adopted in different countries varies, as may be seen from the annexed sketches,



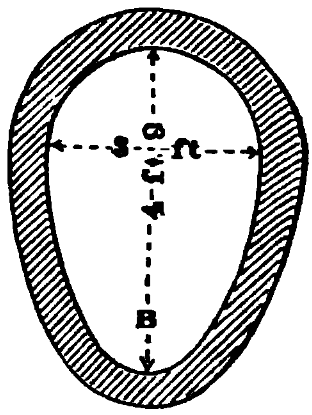


Fig. 296. Sewers. Fig. 297.

of which Fig. 296 represents the section of the main sewers used in Paris; Fig. 297 represents the section lately adopted in our own metropolis. The former is more convenient for the operations of workmen, while the latter is certainly less likely to require cleansing, because the scouring action of the water is made to operate more forcibly upon the materials brought into the sewers.

Whatever be the form of sewer adopted, the dimensions should always be calculated so that it should be able to discharge the maximum quantity it can ordinarily receive without being more than half full. The inclination to be given to house sewers should be, at the minimum, 1 in 144, or 1 inch in 12 feet; that to be given to submains, also at the minimum, 1 in 480, or 1 inch in 40 feet; whilst the inclination to be given to main drains may occasionally be carried as far as 1 in 2300, although it is decidedly preferable to keep within the limits of 1 in 1000. All junctions should be made so that the axes of the secondary sewers should be

portions of circles tangential to the axes of the main sewers, and of the largest radius it is possible to obtain. Side entrances should be formed as closely as possible to the points of intersection of the respective sewers.

The collection and disposal of sewage must evidently, from what has been said before, be entirely guided by the demand for the materials so obtained; and, hitherto, they have all been wasted in England. This is the more to be regretted, because in France, Germany, Belgium, and Italy, the manure so allowed to run to waste in England is found to be highly beneficial, and in our own country all manures are expensive. Near Edinburgh some attempts have been made to apply the sewage, by irrigating meadows with the water from sewers, yet the results there obtained are far from such as are likely to guide us in the selection of any general course of proceeding. The mode of application at Edinburgh is stated, in fact, to be very objectionable, on account of the foul smell given off; and it is also to be observed, that it rarely happens that any extent of meadow land can be found near large towns under the necessary conditions of level to allow of a similar application by mere gravitation. Fairly, the disposal of sewage is the great problem still to be resolved by all parties connected with this branch of engineering; the very injudicious assertions of the advocates of certain theories have hitherto only indisposed the public mind to its examination.

In the case of Edinburgh, the storm and house waters were conducted together upon the meadows irrigated by the sewers. It was found, however, that there was too much manure in the contents of the latter, and it became necessary to form catch-ponds, in order to enable it to deposit; and it must be borne in mind, that in the days when these ponds were formed (1829), the habits of the Scotch people were not such as to cause the bulk of the house manure to find its way into the sewers. The grass from these meadows was

[illegible] times a year, a result so little surpassing [illegible] by ordinary irrigation, that [illegible] any person to incur a large [illegible] similar privileges. In the neigh- [illegible] the same results were obtained, for [illegible] which receive the small [illegible] allow to flow [illegible] first, and [illegible] greater results than [illegible] as grass [illegible] of [illegible] seems to [illegible]

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whenever required; or it is collected in large casks placed in cellars and communicating with the soil pipes. The contents of these different descriptions of cesspools are carried to large lay stalls at Montfaucon and Bondy, and allowed in the former to settle in vast basins or reservoirs, two in number, with a close dam between them, so that one may be used whilst the other is being emptied. The liquid upon the top of these reservoirs is drawn off by sluices, and passes successively into not less than seven other basins, in which it is treated in various manners, for the purpose of causing the deposition of any matters in suspension. The area of the upper reservoir is about equal to 2½ acres superficial, with a depth of about 12 feet; the area of the lower reservoirs is about equal to 13¼ acres. From the last of these the waters are allowed to escape into the main sewer running through the lowlands at the foot of Montmartre. At Bondy, the system of dealing with the manure is in principle similar to that employed at Montfaucon; and in both, the solid matters deposited at the bottoms of the reservoirs are placed in the open air to dry into powder before being used.

Such also is the mode of dealing with sewage in use nearly all over the Continent, and to a certain extent, far too great, even in our own country. Anything more economically absurd, or more injurious to public health, it would be difficult to imagine; and, excepting that some use is made of the manure, the whole system may be cited only as a model to be avoided. Cesspools are always objectionable, because they retain a permanent source of infection, wherever and however constructed. The operation of cleansing them is always disgusting and injurious, whilst the foul exhalations from the depositing reservoirs contaminate the air to a great distance around. Add to this, that, in the operation of drying the deposit, the volatile salts, in fact the most valuable, escape, and our surprise will be increased when we reflect that such monstrous nuisances should be retained in any civilised community.

cut from four to six times a year, a result so little surpassing what might have been obtained by ordinary irrigation, that there is little reason to induce any person to incur a large outlay in order to obtain similar privileges. In the neighbourhood of Milan nearly the same results were obtained, for the waters of the Naviglio Grande, which receive the small quantity of house sewage continental habits allow to flow into watercourses, were found to be too rich at first, and after deposition not to produce much greater results than those derived from any other stream, so far, at least, as grass lands were concerned. Upon corn lands, the application of the comparatively highly-diluted manure of sewers seems to be of very questionable advisability, whilst, at the same time, it is to be observed, that the price of land in the immediate neighbourhood of large towns rarely allows of the cultivation of what may be called bulky crops.

It appears that the common sense of the disposal of sewage matters consists in obtaining the deposition of the fertilising properties they may possess, and in securing them in the most portable form. The great difficulty to be overcome lies in the ammoniacal salts, which no system hitherto proposed has succeeded in obtaining in a permanent form. The use of lime water may cause the precipitation of organic matter, but the salts of ammonia in sewage water usually exist in the condition of the carbonate, and there is not a sufficiently preponderating affinity between the lime and the carbonic acid gas to cause the latter to quit its combination with the ammonia to join the lime. Perhaps the use of the sulphate of lime or the sulphate of iron might be attended with more satisfactory results.

In France, the system adopted in dealing with the whole question of sewage of towns is to separate the rain and surface waters from those derived from water-closets. The latter description of sewage is collected in cesspools, made as impermeable as possible, and from which it is extracted

whenever required; or it is collected in large casks placed in cellars and communicating with the soil pipes. The contents of these different descriptions of cesspools are carried to large lay stalls at Montfaucon and Bondy, and allowed in the former to settle in vast basins or reservoirs, two in number, with a close dam between them, so that one may be used whilst the other is being emptied. The liquid upon the top of these reservoirs is drawn off by sluices, and passes successively into not less than seven other basins, in which it is treated in various manners, for the purpose of causing the deposition of any matters in suspension. The area of the upper reservoir is about equal to 2½ acres superficial, with a depth of about 12 feet; the area of the lower reservoirs is about equal to 13¼ acres. From the last of these the waters are allowed to escape into the main sewer running through the lowlands at the foot of Montmartre. At Bondy, the system of dealing with the manure is in principle similar to that employed at Montfaucon; and in both, the solid matters deposited at the bottoms of the reservoirs are placed in the open air to dry into powder before being used.

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[The most generally approved form of secondary brick sewers is of the type illustrated, Fig. 298, the design of which

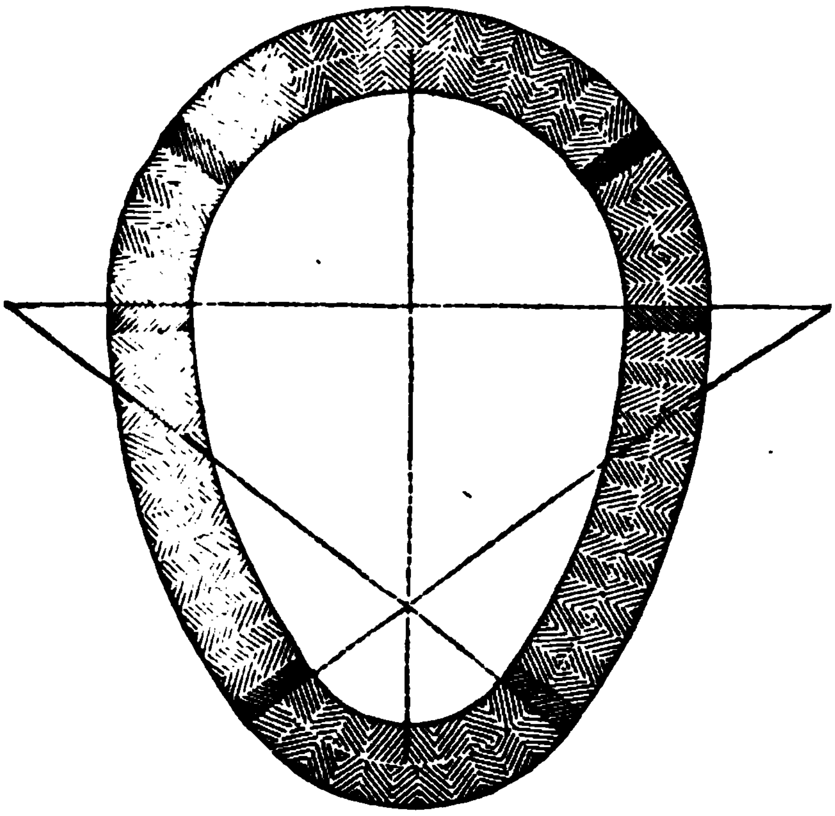


Fig. 298.—General type of Sewer.

is obvious. The main outlet sewer at West Ham, Fig. 299, 4½ feet high and 3 feet wide, was constructed with a cast-iron invert, for a length of about half a mile, at the level of low-water spring tides, and then for 2½ miles, with a fall of 3 feet per mile. The outlet into the Thames is obtained by pumping. The surface of the ground under which the sewer was laid varies from 10 to 12 feet below high-water level; and the subsoil was so porous hat the water could not be pumped below the level shown in the section, Fig. 299. Consequently, an iron invert was devised, upon which cement blocks were laid to above the water-line. The other portions were built in the ordinary manner.

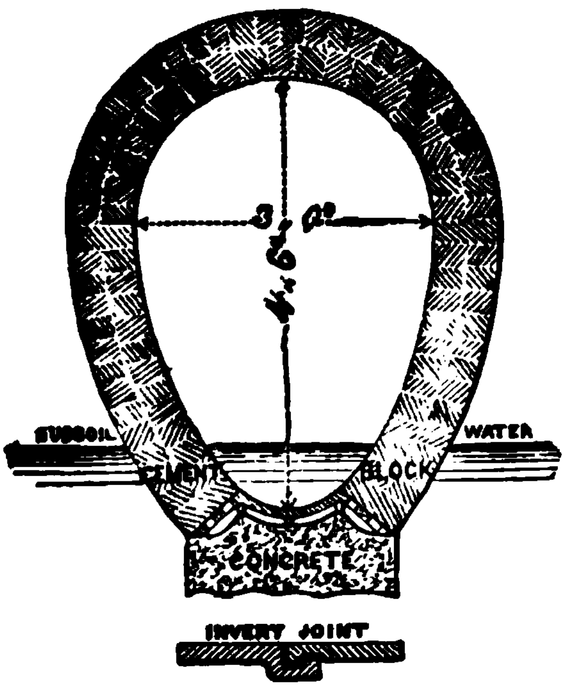
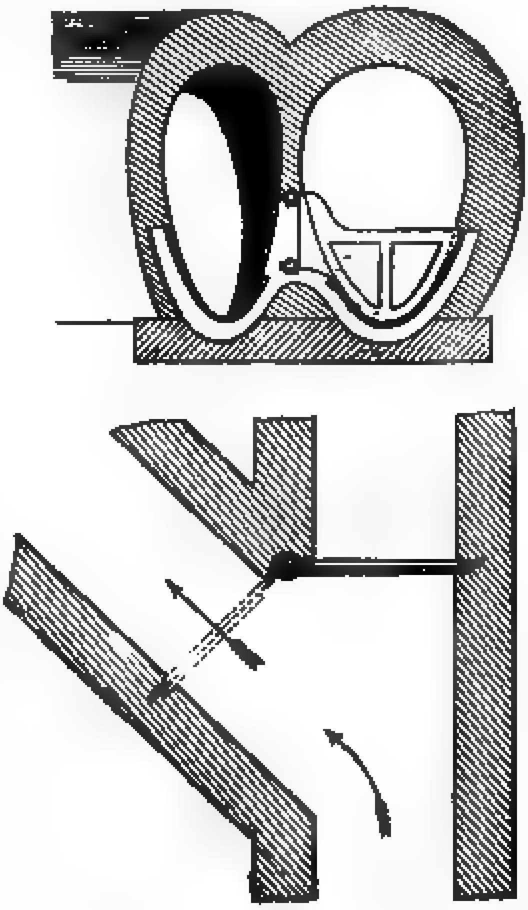


Fig. 299.—Outlet Sewer, West Ham.

Junctions of sewers should be formed at acute angles, in order that the combining currents may not baffle each other. A junction is illustrated in Figs. 300. It is so constructed with a cast-iron gate that, for purposes of flushing, the sewage may be turned into any sewer, or special series of sewers. Placed immediately under a manhole, the sewer may be rendered self-cleansing. The flushing gate is only 15 inches deep, and it answers the purpose of an overflow weir, preventing the sewers during floods from being subjected to pressure, by allowing the surplus to flow into subsiduary outlets, and so equalising the flow of water.



Figs. 300.—Junction of Sewers.

It is dangerous to lay down a system of sewers over the whole of a town without, in the first instance, providing, as part of the scheme, for the fullest and freest ventilation, though it is generally most needed at the upper and higher levels.

In Dundee, the gases of sewers are prevented from escaping through the gullies by valves made of stone, Fig. 301, hung with copper links on one side of a small cesspool. The bottom of the cesspool is 10 inches below the valve-opening; and all heavy matters, such as sand

Fig. 301.—Gully-trap, Dundee.

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line of the bars, with a vertical flange, which, with the stone and brickwork in cement of the cess, for catching the silt, over which it is placed, forms a water-trap for effluvia. A 6-inch pipe is laid to the sewer.

Fig. 303.—Gully-trap, Newport.

The system of ventilation adopted by Mr. Rawlinson is shown in Fig. 305. The manhole shaft is closed with a manhole cover, by removing which access may be had to the shaft when desired. The sewer-gas is compelled to rise through two or more charcoal screens or filters before passing into the ventilating chamber. By a slide at the bottom of the chamber, deposit can be removed. A modification of this system is shown in Fig. 304.

Fig. 304.—Ventilation of Sewers, by Mr. Rawlinson.

Fig. 306.—Ventilation of Sewers: Mr. Rawlinson's System.

A simple and efficient form of street gully-trap used at Preston is shown in Fig. 306.

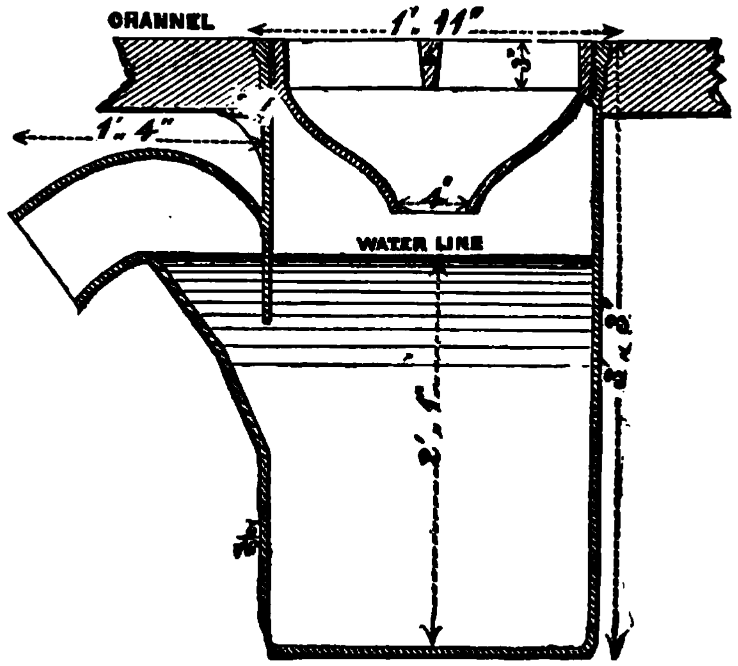


Fig. 306.—Gully-trap, Preston.

The Main Drainage of London.

[The works for the main drainage of London, constructed to the plans of Sir Joseph W. Bazalgette, the engineer, were commenced in 1859, and were practically completed and in operation in 1865. The objects sought to be attained were, the interception of the sewage, as far as was practicable, by gravitation, together with so much of the rainfall as could be reasonably dealt with, so as to divert it from the river Thames near London; the substitution of a constant flow instead of an intermittent flow in the sewers; the abolition of stagnant and tide-locked sewers, with their consequent accumulations of deposit; and the provision of deep and improved outfalls for the extension of sewerage into districts previously, for want of such outfalls, imperfectly drained.

New lines of sewers were laid at right angles to the previously existing sewers, which conducted the sewage direct to the river. The new sewers were laid a little below the levels of the old sewers, so as to intercept their contents and convey them to an outfall 14 miles below London Bridge. For such of the sewage as could not be carried away by

gravitation, a constant discharge is effected by pumping. At the outlets, the sewage is delivered into reservoirs situated on the banks of the Thames, placed at such a level as enables them to discharge into the river at about the time of high water. By this arrangement, the sewage is not only at once diluted by the large volume of water in the Thames at high water, but is also carried by the ebb tide to a point in the river 26 miles below London Bridge; so that its return by the following flood tide within the metropolitan area is effectually prevented.

In answer to the question, What is a sufficient velocity of flow in order to prevent deposits in pipes and sewers, Sir Joseph Bazalgette adopted as the minimum velocity of flow in the sewers a speed of 1½ mile per hour in a properly protected main sewer. This rate of speed he considered to be sufficient when the sewer runs half full, more especially when the contents have previously passed through a pumping station.

To estimate the fall necessary to communicate the velocity of 1½ mile per hour in a sewer, it is necessary to ascertain the quantity of sewage to be carried off. This quantity varies but little from the water-supply with which a given population is provided; for that portion which is absorbed and evaporated is compensated for by the dry-weather underground leakage into the sewers. It was ascertained that a district of average density of population, when wholly built upon, contained 30,000 people to the square mile; and, in districts containing that number, or more than that number, of people to the square mile, the actual numbers were ascertained and provided for; but, in districts where the population was below that number, provision was made for an increase of population up to 30,000 people to the square mile, except over the outlying districts, where provision was made for a population of only 20,000 to the square mile. Now, an improved water-supply, at the rate of 5 cubic feet, or 31¼

gallons, per head for such contemplated increased population has been anticipated.

Experience has shown that sewage is not discharged into the sewers at a uniform rate during the twenty-four hours, nor even throughout the day. Taking a liberal margin beyond the results of actual measurements, provision was made for one-half of the sewage to flow off within six hours of the day. Thus the maximum quantity of sewage ever likely to enter the sewers at various parts of the metropolis was arrived at.

But the rain-fall was also to be provided for. Taking an average of several years, it was ascertained that there were about 155 days per year on which rain falls in the metropolis. Of these, there are only 25 days on which the quantity amounts to $\frac{1}{4}$ inch in depth in the course of twenty-four hours, or the $\frac{1}{100}$th part of an inch per hour, if spread uniformly over an entire day. Of such rain-falls, a large proportion is evaporated or absorbed, and either does not pass through the sewers or does not reach them until long after the rain has ceased. It was concluded by the engineers appointed to inquire—Messrs. Bidder, Hawksley, and Bazalgette—"that the quantity of rain which flowed off by the sewers was in all cases much less than the quantity which fell on the ground; and although the variations of atmospheric phenomena are far too great to allow any philosophical proportions to be established between the rain-fall and the sewer flow, yet we feel warranted in concluding, as a rule of averages, that $\frac{1}{4}$ of an inch of rain-fall will not contribute more than $\frac{1}{8}$ of an inch to the sewers; nor a fall of $\frac{4}{10}$ of an inch more than $\frac{1}{4}$ of an inch. Indeed, we have recently observed rain-falls of very sensible amounts failing to contribute any distinguishable quantity to the sewers." A quantity equal to $\frac{1}{100}$ inch per hour, or $\frac{1}{4}$ inch in twenty-four hours, running into the sewers would occupy as much space as the maximum prospective flow of sewage provided for; so

that, if that quantity of rain were included in the intercepting sewers, they would, during the six hours of maximum flow, be filled with an equal volume of sewage, and during the remaining eighteen hours additional space would be reserved for a larger quantity of rain. It was, then, considered probable that if the sewers were made capable of carrying off a volume equal to a rain-fall of ¼ inch per day, during the six hours of the maximum flow, there would not be more than twelve days in a year on which the sewers would be overcharged, and then only for short periods during such days. Overflow-weirs, to act as safety-valves in times of storm, have been constructed at the junctions of the intercepting sewers with the main valley lines. On such occasions the water is largely diluted, and after the intercepting sewers are filled they flow over the weirs, and through their original channels by the old sewers, into the Thames.

That the sewage should be removed, as much as practicable, by gravitation, so as to reduce the quantity of pumping to a minimum, three lines of sewers, in the nature of the catch-water drains familiar in the fen districts, were constructed on each side of the river, called respectively, the High Level, the Middle Level, and the Low Level Sewers. The High and Middle Level Sewers discharge by gravitation, and the Low Level Sewers discharge only by the aid of pumping. The three lines of sewers north of the Thames converge and unite at Abbey Mills, east of London, where the contents of the Low Level are pumped into the Upper Level Sewer, and whence the aggregate stream flows through the Northern Outfall Sewer, which is carried on a concrete embankment across the marshes to Barking Creek, and there discharges into the river by gravitation.

On the south side the three intercepting lines unite at Deptford Creek, and the contents of the Low Level Sewer are there pumped to the Upper Level; and thence the three united streams flow in one channel through Woolwich to

Crossness Point in Erith Marshes. Here the full volume of sewage can flow into the Thames at low water, but it is ordinarily raised by pumping into the reservoir at that outfall.

The form adopted for the intercepting sewers generally is circular, as combining the greatest strength and capacity with the smallest amount of brickwork and the least cost. This form was, otherwise, practically as good as the elliptical, seeing that, as these sewers only carry off 1-100th of an inch of rain in an hour, whilst the volume of sewage passing through them is at all times considerable, the flow through these sewers is more nearly uniform than in drainage sewers constructed to carry off heavy rain-storms. In the minor branches, for district drainage, the sewer that is egg-shaped in section, having the narrow part downwards, was preferable, because the dry-weather flow of the sewage being small, the greatest hydraulic mean depth, and consequently the greatest velocity of flow and scouring power, is made available with that form of section at the bottom, at the period when it is most required; whilst the broader section at the upper part affords room for the passage of the storm-waters, and also for the workmen engaged in repairing and cleansing.

The High Level Sewer at the north side, commences by a junction with the Fleet Sewer at the foot of Hampstead Hill. It passes thence across Highgate Road, Holloway Road, to High Street, Stoke Newington, at Abney Park Cemetery; thence to Church Street, Hackney, under the North London Railway, through Victoria Park, under the canal, to a junction with the Middle Level Sewer. Up to this point it is a drainage sewer—a substitute for the open Fleet and Hackney Brook main sewers, which have been filled in and abandoned. It is capable of carrying off the largest and most sudden falls of rain. It is about 7 miles in length, and it drains an area of 10 square miles. It is,

for the most part of its length, circular, varying in dimensions from 4 feet in diameter to 9½ feet by 12 feet. It falls at rates of from 1 in 71 to 1 in 376 at the upper end, and 4 feet to 5 feet per mile at the lower end, or 1 in 1,320 to 1 in 1,056. It is constructed of stock-brick work, varying in thickness from 9 inches to 27 inches, and the invert is lined with Staffordshire blue bricks, in order to withstand the scour arising from the rapid fall. One tunnel, from Maiden Lane towards Hampstead, is about half a mile long. Great care was necessary in tunnelling under the New River, its channel being on an embankment where it intersects the line of sewer; also under the Great Northern Railway, at a place where its embankment is 30 feet high, the sewer being 7½ feet in diameter, with 14 inches of brick. Much house property was successfully tunnelled under at Hackney. One house, adjoining the railway station, was under-pinned and placed upon iron girders. The sewer, 9½ feet in diameter, was carried through the cellar without further injury to the house. The sewer is carried close under the bottom of Sir George Duckett's Canal; there is only 24 inches between the bottom of the canal and the soffit of the arch of the sewer. The bottom of the canal and the top of the sewer are here formed of iron girders and plates, with a thin coating of puddle. The whole of the High Level Sewer, including the Penstock chamber, was completed in May, 1861.

The Penstock chamber, at the junction of the High and Middle Levels, at Old Ford, Bow, is 150 feet in length, 40 feet in breadth, and, in places, 30 feet high. It is provided with five large iron penstocks worked by machinery, by means of which the sewage can be diverted at will, either into the two lower channels formed for the discharge of the storm waters into the river Lea, or into the two upper channels constructed over that river, and forming the commencement of the Northern Outfall Sewer. As a rule the lower channels are closed, and the sewage flows through the

two upper channels to Barking Creek. In times of heavy rain, when the water rises to the top of the upper channels, the surplus flows over five weirs, constructed in the chamber, into the lower channels, and discharged by them into the Lea.

The Middle Level Sewer commences near Harrow Road, at Kensal Green, and passes into Uxbridge Road along Oxford Street, Hart Street, and Liquorpond Street, and across Clerkenwell Green; thence by Old Street to Shoreditch, Bethnal Green Road, under the Regent's Canal and the North London Railway, to join the High Level Sewer at Bow. A branch, 4 feet by 2 feet 8 inches, is carried along Piccadilly, with a fall of 4 feet per mile; passes through Leicester Square and Lincoln's Inn Fields to the main line at King's Road, Gray's Inn Road. The main line is 9½ miles in length, and the Piccadilly branch is 2 miles; and there are minor branches and feeders. The area intercepted is 17½ square miles in extent. The fall of the main sewer is from 17½ feet per mile at the upper end, and, by a gradual reduction, 2 feet per mile at the lower end. The sectional dimensions vary from 4½ feet by 3 feet, to 10½ feet in diameter, and 9½ feet by 12 feet at the outlet. Six miles in length were constructed by tunnelling, at from 20 feet to 60 feet underground. The sewer is carried over the Metropolitan Railway by a wrought-iron plate aqueduct of 150 feet span. The sewer is provided with storm overflows into the river.

The Low Level Sewer, besides intercepting the sewage from the low-level area of 11 square miles, serves also as an outlet for a district of 14½ square miles, the western suburb of London, which is so low that its sewage has to be lifted at Chelsea a height of 17½ feet, into the upper end of the Low Level Sewer. This sewer commences at the Grosvenor Canal, Pimlico, and passes to the riverside from Vauxhall Bridge. From Westminster Bridge to Blackfriars it forms part of

the Thames Embankment, Fig. 307; thence, under Queen Victoria Street to Tower Hill, Mint Street, Commercial Road, Bow Common, and under the river Lea to Abbey Mills, where its contents are lifted 36 feet by steam power. It has two branches, one from Homerton and one from the Isle of Dogs. The Isle of Dogs can only be drained by the aid of pumping. The main line is 8¼ miles long, and its branches are 4 miles long. It varies in section from 6¾ feet to 10½ feet in diameter. Its inclination ranges from 2 feet per mile to 3 feet per mile. It is provided with storm overflows into the river.

Fig. 307.—Thames Embankment.

The sewage of the western suburbs, comprising Hammersmith, Fulham, Chelsea, Brompton, Kensington, and other places, is conveyed to the local outfall at Chelsea through a main line of sewers from 4 feet by 2 feet 6 inches to 4½ feet in diameter, with a fall of 4 feet per mile; with branches of from 3 feet 9 inches by 2½ feet, to 4½ feet by 3 feet. The Chiswick sewer is 9¼ miles long; the Fulham sewer is 1 mile 720 feet, and the Acton branch is 1½ miles, with minor branches.

The northern outfall sewer is a work of peculiar construction; for, unlike ordinary sewers, it is raised above the ground by an embankment. It consists of two brick sewers or culverts, each 9 feet by 9 feet, side by side, with upright sides, semicircular crowns, and segmental inverts. These are built upon a solid concrete embankment, carried through the peat soil up from the gravel. Concrete is also carried up at a slope of 1 to 1, so as to form an abutment for the sides of the sewers. The whole structure is covered with an earthen

embankment. The "line," as it may be called, is carried by aqueducts over rivers, railways, streets, and roads. The aqueducts consist of two wrought-iron culverts of the same section as the brick sewers, upon which a roadway is formed, with parapet walls, the whole being supported by three wrought-iron girders. From the Abbey Mills, three parallel sewers of the same dimensions as those just noticed are carried to the outfall at Barking Creek. Gates and overflow weirs are formed in the line of these culverts, enabling the sewage to be turned into either or all of them at will, and preventing any one from being at any time overcharged. For a distance of about 1½ miles at the lower end of the sewer, the depth of peat in the marshes was so great that, instead of carrying up a solid embankment, the sewer was carried on piers and arches. The bank is 40 feet wide at the top, and is in some places 25 feet above the level of the marshes.

The reservoir at Barking is 16¾ feet deep, in four compartments, covering an area of 9½ acres.

At the south side of the river, the High Level Sewer and its Southern Branch correspond with the High Level and the Middle Level Sewers on the north side. The main line commences at Clapham, and the branch line at Dulwich, together draining an area of 20 square miles. Storm-waters are discharged into Deptford Creek, whilst the sewage and a limited quantity of rain flow, by four iron pipes, 3½ feet in diameter, laid under its bed, into the outfall sewer. The two lines come together in the New Cross Road, at the branch 10 feet above the main line. They come to a level at Deptford Broadway.

The branch is 4½ miles in length, of which 1,000 feet were constructed in tunnel at depths of from 30 feet to 50 feet. It varies from 7 feet in diameter to a form 10½ feet by 10½ feet, with a circular crown and segmental sides and inverts. Its fall varies from 30 feet per mile at the lower end to 2½ feet per mile at the lower end.

The main line varies in size from 4½ feet by 3 feet at the upper end, to 10½ feet by 10½ feet, like the branch.

The outlets of the two sewers, 10½ feet by 10½ feet, are each fitted with two hinged flaps, one above the other. The lower flap is usually fixed close, so as to form a dam to drive the waters through the iron pipes in the outfall sewer; but the upper one hangs free, in order to serve as a tide flap and allow the sewage to pass into the Creek, when it rises to a sufficient height. In cases of heavy floods, the lower flaps can be opened.

The main line falls 53 feet, 26 feet, and 9 feet per mile at the upper end, to Brixton Road, and thence 2½ feet per mile. The sewer is in brickwork, varying from 9 inches to 22½ inches; one half, the invert, being in Portland cement, and the remainder in blue-lias mortar.

The Low Level Sewer begins at Putney, and passes through Battersea, Nine Elms, Lambeth, Newington, Southwark, Bermondsey, Rotherhithe, and Deptford, comprising an area of 20 square miles. It is 10 miles long. In size it varies from a single sewer 4 feet in diameter at the upper end, to two culverts, each 7 feet by 7 feet. Its fall is from 4 feet to 2 feet per mile, and the lift at the outlet is 18 feet.

A branch, 2 miles, is laid from St. James's Church, Bermondsey, to Deptford, with a fall of 4½ feet per mile.

From Deptford the sewage is conducted by the southern outfall through Greenwich and Woolwich to Crossness Point, in Erith Marshes—a distance of 7¾ miles—entirely underground. It is 11½ feet in diameter, in brickwork generally 18 inches thick, with a fall of 2 feet per mile. The bottom of the sewer is 9 feet below low water at the outlet into the river, so that it can discharge at and near low water by gravitation. But, ordinarily, the sewage is discharged by pumping into the Crossness Reservoir, which is 6½ acres in extent. The tunnel through the chalk under Woolwich is the principal feature of the work.

The bricks used in the works were mostly picked stocks and Gault clay bricks; and the inverts were occasionally faced with Staffordshire blue bricks. The brickwork was laid in blue-lias lime mortar, in the proportion of 2 of sand to 1 of lime for the upper part, and Portland-cement mortar in the ratio of 1 to 1, below. A very considerable length was laid entirely in cement.

The total cost of the main drainage works is stated to have been £4,100,000. There are about 1,300 miles of sewers in London, and 82 miles of main intercepting sewers. The total pumping power employed is 2,380 nominal horse-power. The sewage on the north side of the river amounted, in 1865, to 10 millions of cubic feet per day, and on the south side to 4 millions per day; but provision has been made for 11½ millions and 5¾ millions respectively, in addition to 28½ millions of cubic feet of rain-fall per day on the north side, and 17¼ millions on the south side: making a gross total of 63 millions of cubic feet per day, equivalent to a lake fifteen times as large as the Serpentine in Hyde Park.

Drainage of Paris.

When, in 1808, the subject of the main drainage of Paris first received a systematic investigation, the "great" or main drain was only a large natural ditch—the brook of Menil-montant converted into a ditch or open sewer. In 1740, this ditch was walled and arched over. It was assumed, in 1808, that the drains should empty themselves into the river Seine, following the undulations of the streets. The river, within the limits of the city, was made the main receiver of all the sewage. From both banks and from the central islands all outlets poured directly into it, and at the end of 1837 there were probably forty independent mouths. By the more recent developments, all these discharging mouths were, with three exceptions, abandoned, and longitudinal intercepting drains of a large section, running parallel to the

river, were substituted, and the drainage was collected into four points of discharge, namely, Asnière, Chaillot, the Isle of St. Louis, and the Isle of Nôtre Dame; in addition to which, in 1863, 217 miles of main sewers had been constructed in Paris. In January, 1874, a total of 356 miles had been constructed; and it was then estimated that nearly 287 miles —mostly in the suburbs near the fortifications—remained to be executed, in order to complete the whole system.

The drains are confined to the carrying off of rain-water and household-water. Night-soil has no connection whatever with the drains. It is carted away from the city and deposited at appointed places. Road scrapings find their way into the drains. Besides, the sand which is stored in heaps along the streets for purposes of repair is converted into a mud which, with the road-scrapings, is not uncommonly swept into the drains. In many cases, the inclination of the drains is so slight that they can hardly carry off the solid matter, which, in fact, is deposited, and must be cleared away by mechanical means.

The main intercepting sewer of the south side, "Grand Egout Collecteur de la Rive Gauche," is of the type No. 3, Fig. 308. It commences at the Boulevard St. Marcel, Jardin des Plantes, and terminates at the siphon of the Pont de l'Alma, 10 feet above the lowest water-level of the Seine. It is 6,015 yards, or 3·42 miles, in length, with a fall of 1·8 feet to the mile, or about 1 in 3,000. The branch sewers are mostly of the section shown in Fig. 309.

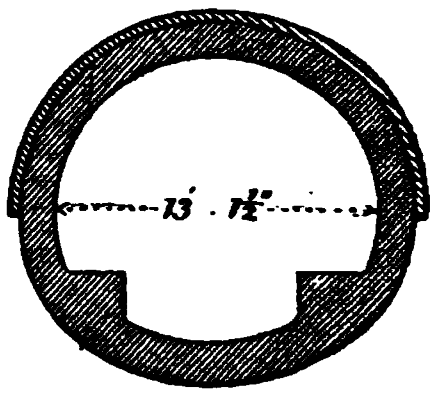


Fig. 308.
Intercepting Sewer, Paris.

The main intercepting sewer of the north side, "Collecteur Générale de la Rive Droite," is to the section, type No. 1, Fig. 310. It commences at the Boulevard Sebastopol, and terminates at Clichy, opposite Asnières. The fall varies from 1 in 798 to 1 in 2,766.

The "Collecteur de la Bièvre" is a prolongation of the main sewer of the south side, of the same section as this sewer. It commences at the outfall of the siphon Pont de l'Alma, and terminates by a junction with the main sewer of the north side at a distance of 601 yards from its outfall into the river. It is nearly 3 miles in length, and the fall varies from 1 in 1,925 to 1 in 2,951.

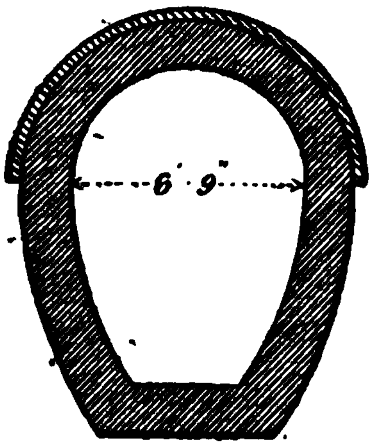


Fig. 309.—Branch Sewer, Paris.

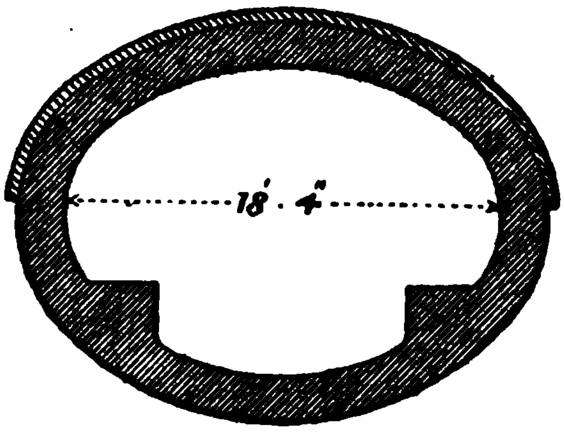


Fig. 310.—Intercepting Sewer, Paris.

In streets of a width greater than 65½ feet, double lines of sewers are constructed, one under each footpath, to limit the length of the branch drains, which are laid in at the expense of the proprietors. The principle of the design of the drains of Paris, it may be noted, differs materially from that of London and other English towns. In Paris, the oval form is followed, but the invert, or "radier," is nearly flat, of a width varying from 16 inches to 24 inches, in order to afford footing for the workmen engaged in cleansing the sewers. For the same reason they are made of considerable height. The smallest section of drain that has been constructed was 5½ feet high, and 2½ feet wide at the springing of the roof.

The deposit in the sewers is removed by mechanical means. In the smaller sewers, where small flushing-boards and iron half-gates are occasionally used, the deposit is pushed forward into the larger sewers by wooden hand-scrapers. In many of the smaller types, hand-tipping trucks convey the refuse into the larger sewers. In the larger sewers, sluice-gates, hung by chains to the crown of the sewer, are frequently

lowered, in order to back up the water so as to float the flushing-boats by which the cleansing is effected.

The siphon of the Pont de l'Alma, for conveying the sewage of the south side to the north side across the Seine, is composed of two lines of wrought-iron tubes 3¼ feet in diameter, ¼ inch thick. It has a minimum head of 1·70 feet, the difference of level between the south side and the north side; but the head can be increased to 7·8 feet. An effective system of scouring can thus be practised. A wooden ball 33½ inches in diameter, weighing 187 lbs., is sent through the tubes once a week. As it is about 6 inches less in diameter than the tube, it rolls along the crown of the tube; and should its course be arrested by deposit an additional scouring takes place under the ball, and the sediment or accumulation is dislodged and removed.

The greater part of the excremental matter is drained into cesspools not connected with the sewers, and it is pumped thence into cylindrical iron carts. These, when filled, are hermetically sealed and conveyed to the several depôts. The more modern system is that of the "*appareils diviseurs*," or "*tinettes filtres*"—sheet-iron cylinders for separating the fluid from the solid portion of the excreta. The fluid is either discharged into a cesspool or is sent into the sewers.

Drainage of Hamburg.

The sewerage of Hamburg, designed by Mr. W. Lindley, is worthy of notice. It was executed upwards of thirty years ago. The most striking feature of the system is the means of constantly flushing the sewers, which are kept so clean that it is rarely necessary to send any men into them except for the purpose of making repairs. The forms and dimensions of the sewers vary with their inclinations, as well as with the quantities of sewage which they were designed for carrying off. The steep sewers are cylindrical, from

15 inches to 20 inches in diameter; their fall varies from 1 in 15 to 1 in 150. Four or five miles of such sewers have been laid. For long main lines, where such rapid falls could not be obtained, oval sewers of larger dimensions were built, sufficiently large for men to pass through, being 4½ feet high and 2½ feet wide. This class of sewer was not made with less inclination than 1 in 500 in the upland districts of the city, where the quantity of water to be obtained for flushing is dependent upon the waterworks. But in the marsh districts, where the water for flushing is obtained from the river Alster, sewers of this description are laid as nearly level as an inclination of 1 in 3,000. The marsh level would not allow of a greater fall, and the unlimited flushing power derived from the river rendered the adoption of such low gradients quite unobjectionable. Every house has its drain-pipe discharging directly into the sewers, without any cess-pool. Where there is fall enough, a 6-inch cast-iron pipe is carried directly into the opening built in the sewer to receive it; but where there is less fall, the brick drains, of 12 inches or 15 inches in diameter, receive the soil-pipes within the house and discharge the contents into the main sewer.

The larger classes of sewers follow each other in succession as the number of smaller sewers are united and the quantity of water to be carried off increases. The largest sewer is 5 feet wide and 6 feet high. During heavy storms of rain the quantity of water delivered into it from all its tributaries is so great that it has, on a few occasions, been filled nearly to the soffit.

The whole of the upland sewers were laid out so as to act as catch-water drains, and thus to separate the upland waters from the marsh sewers. This arrangement was necessary, because the water in the Elbe remains at times, from 24 to 36 hours, above the cellars of the houses. The upper ends of all the sewers are connected either with the river

Alster, which affords a head of 13 feet of water, or with the canals by which the city is intersected, or with other high-lying sewers—the latter forming reservoirs supplied from the waterworks. Thus there are not any dead-ends, and all the sewers can be thoroughly flushed from one end to the other. By this arrangement, it is true, additional outlay was incurred for extra lengths of sewer to form the connections with the water-heads, and for penstocks and flushing-gates, to turn the stream in the direction required through the sewers. But the general result of this through system of flushing was great economy in working, and it was proved that the cheapest means of getting rid of the matters discharged into the sewers was to dilute them with large quantities of water, and thus to flush them away.

There are two outfalls for the sewage of Hamburg, one for the upland waters, and the other for the lowland waters. The two discharges are delivered into the main stream of the river Elbe at a point below the city and the harbour, so selected that steam-engines may be placed to pump the sewage water over the lands of the adjoining district, when such a proceeding may be deemed advisable.]

CHAPTER XI.

RECLAMATION OF LAND.

It frequently happens that large tracts of alluvial deposits are found at the mouths of rivers, which are alternately covered or left bare by the tides, and which, generally speaking, continue to increase until they attain such a height as only to be affected by the spring tides. These banks then become covered with a species of marine vegetation, and are cut up into innumerable small creeks, which, at the low water, serve as channels for the inshore streams. Many banks of this description have been reclaimed from the tidal action, both in our own country and in Belgium and Holland, with such signal advantage, in many cases, as to cause regret that others should still remain unproductive.

The works usually required to reclaim these foreshores consist—firstly, of an embankment forming an enclosure to protect them from the sea, which must be able not only to resist the hydrostatic efforts of the external waters, but also the more destructive action of the waves and the currents; secondly, of the system of drainage of the enclosed lands, including under this head occasionally the arrangements for introducing waters charged with fertilising matters, an operation performed in some districts, and known locally by the name of "warping."

The enclosure banks are made, generally speaking, from 2 to 4 feet above the high-water line of the equinoctial spring tides, with a minimum width of from 3 feet 6 inches to 7 feet

at the crown. The outline of the bank in plan must depend upon many local circumstances; but, theoretically, it will be found to offer the greatest resistance to the normal action of the waves if it be convex seawards, whilst the stability of the material, if it be executed in stone rubble, will be the greatest if the outline be concave. Whatever be the form given in plan, it must always be borne in mind that no sharp internal angles should be allowed, and that every projection must be joined into the body of the work by gentle curves of the largest possible radius.

The best form of the sea slope is a subject still much in discussion amongst engineers. On the shores of Holland and Belgium the practice has been, for many years, to make it rectilinear, and inclined at a small angle to the horizon. Although these slopes have succeeded in some positions, there are others in which the results obtained have been precisely of an opposite character, and in which it would appear that a vertical wall would have been preferable. Again, many distinguished engineers are of opinion that the best form to be given is one similar to the outline the materials themselves would assume if left to arrange themselves by natural causes; whilst latterly Colonel Emy has advocated, with considerable ability, the theory that a concave transverse section was the most fitted to resist the action of the ground waves.

Long fore slopes possess the advantage of allowing the employment of any sand, or other similar materials; they offer the least resistance to the action of the sea, and are precisely the less exposed to injury in proportion as their inclination is greater. It has been observed that the destructive action of the sea exercises its greatest effect about the level of the lowest high tides of the neaps. But if these long slopes possess some advantages, they are accompanied by corresponding disadvantages; for they conduct the waves to much higher points than they would otherwise reach, and it is not always that either the materials at hand or the space

disposable are such as to allow of their economical execution, to which consideration after all the decision as to works of this description must be referred.

Vertical enclosure walls occupy the least space, and expose the smallest surface to the action of the waves; and these again, instead of breaking upon the shore, are reflected towards the open sea. But walls of this description must encounter the maximum effort of the waves, wherever these do strike, and their recoil must act very injuriously upon the footings, unless they be of a very resisting description. The concave walls recommended by Colonel Emy have not yet been tried in a sufficient number of cases to justify any definite conclusions as to their merits; but they are in many cases objectionable on the score of the ground they require, and the great expense, not only of the first cost, but of the repairs.

The reasons which should influence the choice of the form to be given to the sea slope of an embankment may be resumed as follows. 1. It will be influenced by the main direction of the winds, waves, tides, and currents, which should be made to strike the bank as nearly as possible in a direction normal to the surface of the facing. 2. By the materials to be procured in the neighbourhood. 3. By the surface of land which can be devoted to the formation of a bank. 4. And principally by the commercial considerations affecting the original execution, the maintenance, and the value of the whole operation.

The inner slope of the banks will depend upon the materials of which it is composed; and at its foot a catch-water drain must be formed to collect the waters falling upon the enclosed land, and to conduct them to the outfall. The Dutch engineers usually make the slope about 5 to 1, and they form a roadway about 20 feet wide between its foot and the edge of the catch-water drain. When the bank is formed of mud or silt, it is necessary to carry up in its centre a core of sand or other hard substance, to prevent rats or moles from boring

through it; and means must be taken to cover the exposed surfaces with vegetation of a character to bind together the materials of which the bank is made.

Fig. 311.—Reclamation of Land.

The land waters collected in the outfall drain are let off by means of sluices, whose apertures will be regulated by the quantity to be discharged, and the duration of the period in which the flow can take place, as well as by the head of water which may exist at the commencement of the discharge. Upon the sea-coast the intervals between the tides recur with great regularity; but in the upper portions of river-courses the casual floods are likely to prevent the discharge during periods of variable duration, so that in many such positions it is very probable that the reclaimed lands may be partially, or entirely, flooded on all such occasions: the cultivation to be adopted must be regulated with a view to these contingencies.

The simplest mode of closing the outfall drain is by a sluice upon hinges, fixed at the outer end of a culvert, in wood, masonry, or iron, passing through the body of the bank. The floor of this aqueduct is placed at the level of the bottom of the catch-water drain, and it has an inclination outwards. So long as the head of water upon the outside of the sluice is greater than that upon the inside, it will remain closed; directly the waters upon the outside have fallen so as to form a sufficient head upon the inside to overcome the friction of the hinge, the sluice will open and give passage to the waters. It is, however, advisable that a sliding gate working in a valve be placed behind the hinged sluice, to

guard against the possibility of accidental derangements of the latter.

Another description of gate frequently used in these works is the gate working upon a vertical axis and shutting against a rebate, in which the areas of the two portions of the gate are made unequal. When the waters on the outside are higher than those on the inside, the gates are pressed against the rebate; when the opposite conditions occur, the gates open and afford a passage to the land waters. Sometimes in large gates of this description where two leaves are employed, they are made to meet at an obtuse angle, like the leaves of a lock gate.

The system of warping is much adopted on the banks of the Humber in our own country, in Tuscany, in the valleys of the Chiana and of the Rhone; and, indeed, the cultivation of the valley of the Nile is but an illustration of it upon a very extensive scale. It is founded upon the principle that all rivers carry, in their downward course, the earthy matters they derive from the lands surrounding their water-shed. The waters so charged are allowed to flow over the land to be warped, and they are retained upon it until the earthy matters are deposited, when they are allowed to run off by means of surface weirs.

It is usual to surround land proposed to be thus treated by an embankment, in which are placed the inlet sluices, at the lowest level. The water enters through these sluices at the highest point of one tide, and is retained during the interval between two successive tides; to be then run off entirely, even from the ditches, before the influx of the next. Upon the banks of the Humber it is considered that the most beneficial efforts are produced by the execution of this operation between the months of June and September; the embankments are made from 3 feet to 7 feet high, and it is usually calculated that a sluice, with a clear water-way about 6 feet high and 8 feet wide, will suffice to warp a surface of from

60 to 80 acres. In this district it is found that the warped lands are at first cold and raw, and that they require a peculiar treatment for agricultural purposes.

The quantity of sediment brought down by the rivers falling into the Humber is enormous. Lord Hawke stated, in his Report on the Agriculture of the West Riding, that one tide would deposit an inch of mud, and the source from whence it is derived is still a matter of great uncertainty. At its mouth the Humber is as clear as most rivers, and the floods from the upper countries, so far from increasing the quantity of matters in suspension, on the contrary, exercise a very injurious effect upon them. In the driest seasons and the longest droughts it is found to be the best and most plentiful, and produces its effect totally irrespective of the subsoil. In fact, a new soil is formed, and the operation of warping differs in this respect from ordinary irrigation, which acts by improving the soil already existing.

CHAPTER XII.

IRRIGATION OF LAND.

IRRIGATION is a branch of Hydraulic Engineering, which has not received the attention from the public in our country which its advantages and results would appear to warrant. In France, Spain, Italy, Egypt, and India, however, it has for ages engaged the attention of agriculturists and governments, and immense national works have been executed to insure its successful application. In Sweden and Northern Germany irrigation works have been carried out to a great extent; but, inasmuch as it appears that the more temperate zones are the most adapted to insure satisfactory economical results, and that practically it is advisable to confine irrigation to countries situated between the 25th and 27th parallels of latitude, in the northern hemisphere at least, it is only exceptionally that works of this description have been executed beyond those limits.

All waters are not equally fitted for the purpose of irrigation, and a certain degree of care is required in their selection, whatever be the description of cultivation to which it may be proposed to apply them. Those which flow from forests, peat-mosses, or contain large quantities of the oxide of iron, are, if not positively injurious, at least but little adapted to this use. Those waters are the best which have been the longest exposed to atmospheric influences, or which may have traversed fertile lands able to communicate some of their properties. It is on this account that streams

flowing through towns and villages are the most desirable. provided always that they do not become too highly charged with sewage. The waters flowing from granitic or primary rocks are often more advantageous than those from the secondary formations, particularly if these consist of the magnesio-calcareous deposits. The waters from the argillo-calcareous rocks, or marls, possess an intermediate character; but as the condition to be fulfilled by any water used in irrigation is that it supply the deficiencies of the soil traversed, it may frequently happen that calcareous waters may be the most advantageous. A very simple criterion, however, exists, by which the adaptation of the waters of any particular stream to these purposes may be judged; it consists in the nature of the vegetation in the natural bed. If this be covered with a luxuriant vigorous herbage, of a good quality, the water may safely be considered to be fitted for the proposed use.

The description of soil which derives the greatest benefit from irrigation is that which is the most permeable and the most easily warmed. Compact clay lands gain the least, because they absorb the heat necessary to insure that the water should produce its greatest effect with difficulty; and, as they are very retentive, the evaporation from them cools the ground to a serious extent. The nature of the subsoil is, however, able to modify very considerably the practical application of these rules.

The greatest advantage, economically, is derived from the irrigation of what are called natural and artificial meadows, in the higher latitudes at least; more southerly, garden grounds, rice fields, and even sugar plantations, are found to derive immense benefit from the process; and, indeed, it is hardly too much to say that the difference between fertile land and barren wilderness in warmer countries may be attributed to the presence or absence of some means or description of irrigation. The terms *natural* and *artificial* meadows, used

above, are to be understood as referring to the nature of the vegetation only; the former, the natural meadows, being those in which plants of the natural order of the *Gramineæ* are principally grown, such as the *Phleum pratense*, *Lolium perenne*, *Festuca sylvatica*, *Poa pratensis*, &c.; whilst the latter, the artificial meadows, produce plants of the order *Leguminosæ*, which require to be sown every year, such as the *Medicago sativa*, *Trifolium pratense*, *Vicia sativa*, &c. Of these descriptions of meadows, again, the best results are obtained from those called *natural;* and, as they are almost the only ones adopted in our country, the following remarks will be confined to them.

The period of the year in which the water should be poured over the land will vary with the latitude and the purposes to which it is to be applied. In England it is used sometimes for the express purpose of protecting the vegetation from the effects of frost, and is therefore applied in winter; but if it be desired to retain the matters in suspension in the waters, they should be used in the later part of the autumn and in the early spring, because it is at those epochs of the year that rivers are the most charged, under normal circumstances. The usual practice in the south-west of England is to irrigate through the months of October, November, December, and January, from fifteen to twenty days at a time, without intermission. At the expiration of each of these periods the ground is left to dry during five or six days. If a slight frost should occur, the water is again immediately turned on, but the ground is left dry if there be any probability of a long-continued and severe frost. In February the length of the periods of irrigation is diminished to about eight days, and care is taken to shut off the water early in the morning, so as to allow the ground to dry during the daytime, and thus obviate any danger from the light frosts at night. In March the same precautions are observed, and the periods of irrigation gradually diminished, in such pro-

portions that the ground shall be thoroughly dry before the end of the month. The meadows are then depastured during the month of April by sheep and lambs, and eaten barely down before May by a heavy stock. After that the grass is allowed to stand for hay, and in some districts it is usual to irrigate for a week before it is so left; but, as an invariable rule, it appears that when the grass is 2 inches high no more water is applied.

Occasionally the lands are irrigated after the crop of hay has been carried; but it is asserted that the grass of the aftermath is, under such circumstances, very injurious to sheep. Grass lands irrigated in summer are known to produce the rot in those animals, though cattle are not affected in a similar manner. It is known, also, that if the purest water remain upon land for any length of time, especially in spring or summer, it deposits a species of white scum, of the consistence of melted glue, which acts very injuriously upon the qualities of the grass.

Very little is known with respect to the quantity of water required to irrigate a definite surface; and, indeed, this must depend upon many circumstances connected with the latitude of the district and the nature of the subsoil. In the south of France, it has been calculated, an acre of meadow land would require about 1,200 cubic feet of water per day during the season for irrigation; but there the land is very light, and the ground, owing to the summer heats, is very dry. In England it is almost certain that, even upon tolerably light lands, it would not be necessary to employ much more than half the above quantity. In the county of Gloucestershire the practice is to allow a stream of 2 inches in depth to flow over the surface, and to dress the latter with a fall of half an inch to a foot from the feeder to the drain.

The primary conditions for the establishment of a system of irrigation are, that a copious supply of water exist at all times, and that the land to be irrigated should present such

a configuration as to allow the waters to flow over it with a regular current, and to insure a perfect discharge of the water after it shall have passed over the land.

The water may be poured over the land either by means of a dam across the whole width of the channel, or by a lateral deviation, according to the water privilege of the landowner. The former course is preferable wherever it can be adopted, because it enables the water to be penned back, and thus poured over a greater surface and upon higher points; but it is necessary to pay particular attention to the effects of such a dam upon the flow of the stream, in order to avoid flooding the lands of neighbours. It must be borne in mind that the top water line of any intercepted stream is never horizontal, but that it assumes a hyperbolic curve, which may be considered to join the natural declivity at a distance varying with the inclination of the bed.

In Spain the waters for irrigation are, in many cases, obtained from artificial reservoirs, formed by throwing dams across the narrow gorges of deep valleys; and the various reservoirs constructed in many districts of England might be made to perform the same office. The construction of the transverse dams in such works is a matter of vital importance in every sense of the word, as was lately exemplified in the case of the Holmfirth catastrophe; and it is impossible to dwell too much on the necessity for the careful construction of the foundations, so as to prevent any infiltrations. When these dams are formed of earthwork, the crowns should be made of a width equal to half the height, and the base be at least three times the height. It is safer to make the principal slope on the inside, towards the water, and to form it in steps; and it would also be preferable to make the dam convex inwards. The top should be at least 2 feet above the highest water line; two sluices should be placed near the bottom, one to draw off the water, the other to allow the reservoir to be cleansed; and overflows should be formed to

prevent the waters ever rising to the top of the dam itself. If the streams flowing in be charged with very large quantities of matter in suspension during the rainy seasons, it may also be necessary to construct depositing basins to receive the mud and sand they bring down.

The construction of large reservoirs has been treated at some length by the author, in an article upon Inland and River Navigation, inserted in the "Aide Mémoire to the Military Sciences." The reader is referred to it for a description of the precautions to be taken and the lessons to be derived from similar works already executed. It may suffice here to state, that it is indispensable that the strata of the valley, in which it may be proposed to construct a new reservoir, should be naturally impermeable or rendered so by art, and that they should not allow of any unequal settlements in the dam. Care must also be taken to prevent the detrusion of the dam, by stepping its foundations and by avoiding any horizontal joints traversing the whole thickness.

When a supply of water has been secured, the next operations will consist in the disposal of the ground in such manner as to insure, firstly, that the water arrive by the culminating points; secondly, that it be distributed equally and with a proper velocity over the whole surface; and thirdly, that it be collected into the outfall drains directly it shall have passed over the land to be irrigated.

There are two systems for preparing the land for this purpose, which are known in the south-western counties by the names of bed-work and catch-work irrigation. In *bed-work* irrigation, the land is thrown into beds or ridges, in directions at right angles to the main feeder as far as possible, although that precise arrangement is not absolutely necessary. In *catch-work* irrigation, ditches are made at distances below each other, across the declivity, to catch the water flowing from the top of the field and distribute it again and again over the land. The former system is more expensive,

but it is far more equally successful than the latter; because, evidently, the land receiving the water at the point where it first leaves the stream, must retain a larger portion of the fertilising materials it may afford than those parts receiving the water, as it were, at second-hand. Catch-work irrigation, in fact, should never be resorted to but in those positions where the declivity is too great to admit of the troughs or distributing gutters being made to point down the descent.

The beds and ridges are so disposed that a ridge may be formed in the centre, with a slight longitudinal fall, and the ground on either side slope away to a drain intended to receive the waste waters. The channels or floating troughs are placed upon the ridge, and communicate with the main feeders or conductors. Their inclination is usually made about 1 in 500, and practically their length appears to be confined to about 70 yards. When the surface to be irrigated exceeds that width, it is usual to form fresh conductors, so that the water should not flow over the land for a greater distance than 70 yards without being again carried into the natural bed. The usual dimensions of these channels are about 20 inches in width at the junction, and 12 inches at the end. The inclined planes on either side of the trough have inclinations varying with the nature of the soil and the supply of water. In light and absorbent soils they require to be slight in order that the water may remain long on them and not to scour the ground; in compact heavy lands, on the contrary, they should be greater. The limits of variation are between 1 in 1,000 and 1 in 100, according to the nature of the ground. The width of the planes, also, is dependent upon the same considerations. The more compact the nature of the soil, the wider must be the planes, because the water can flow over a greater surface without being absorbed; whilst in open porous soils the width must be diminished. Upon the former a width of about 130 feet may occasionally be adopted; upon the latter 40 feet is the

usual width. When the bed falls in one direction longitudinally, the crowns or ridges should be in the middle; if they fall laterally and longitudinally, the crowns should be made towards the upper side; and in either case they should project slightly above the planes.

The dimensions and inclinations of the drains at the foot of the planes must be made sufficiently great to insure the speedy and effectual removal of the water.

The inclination and sectional area of the conductor must be regulated by the number and position of the side floating troughs it may be required to supply, taking into account the quantity which may be absorbed by the earth or lost by evaporation during the passage through the conductor. The latter source of loss may be diminished by confining the width of the canal within the smallest limits possible. If the river carry down much extraneous matter it is advisable to give a tolerably sharp fall to the conductor, in order that it may not be deposited in the latter. An inclination varying between 1 and $1\frac{1}{4}$ in 10,000 will be found sufficient for this purpose in the majority of cases. It is also desirable that the conductor be made as narrow as possible, in order to occupy the smallest quantity of land.

The formulæ for ascertaining the dimensions of the channel are, $Q = S\ V$; in which Q = the quantity to be *supplied*; S = the sectional area; and V = the velocity. By transposition, this formula becomes $S = \frac{Q}{V}$, and V may be ascertained by the formula, given by Playfair, from De Prony, $V = -\cdot 154118 + \sqrt{\cdot 023751 + 32806\cdot 6 \times RI}$; in which R is the hydraulic mean depth, or a quantity obtained by dividing the area of the transverse section, expressed in square inches, by the perimetre, or boundary of that section minus the breadth of the surface, expressed in lineal inches; and I = the sine of the inclination. As the angles formed with the horizon are infinitely small, in these operations it is

generally found to be sufficient to substitute the rate of inclination for the more theoretically correct term of the sine. It is hardly to be supposed that the class of workmen who usually direct irrigation works make many calculations of this kind; nor, with the superabundance of water we have in England, is it often necessary to consider the precise dimensions required. It may happen, however, that it will be found desirable to execute irrigation channels in countries where water is more valuable. In Gloucestershire it is usual to make the conducting channel, for a surface of 300 acres, about 15 feet wide by 3 feet deep. The distance between the feeders and drains, in cold swampy land in that county, is also ordinarily confined to about 4 or 5 yards.

Hutches or sluices should be placed at the points where the conductor communicates with the stream, or where the floating troughs branch off from the conductors, so as to regulate the admission and distribution of the water at any period. The most important of these is the hatch at the mouth of the conductor, which will require to be of considerable strength in order to resist the efforts of any sudden freshets; for if these should occur when the crop is in a forward state, and bring down waters charged with much sedimentary matter, they may produce very disastrous effects. The floating troughs themselves may be closed by movable dams, or merely by pieces of turf laid across the mouth.

All the above remarks must be considered as only possessing a very general application, and as being susceptible of variation according to local circumstances. Thus the inclination frequently given to the main conductors in the mountainous districts of the Alps, Tyrol, Savoy, Dauphiné, and Pyrenees is $\frac{1}{500}$; whilst in the private canals lately executed in Piedmont and Lombardy it varies from $\frac{1}{1600}$ to $\frac{1}{3600}$; and in La Provence it varies from $\frac{6}{10000}$ to $\frac{9}{10000}$. It would appear that in mountainous countries, therefore, the higher limit may be adopted; but that if the inclination approach

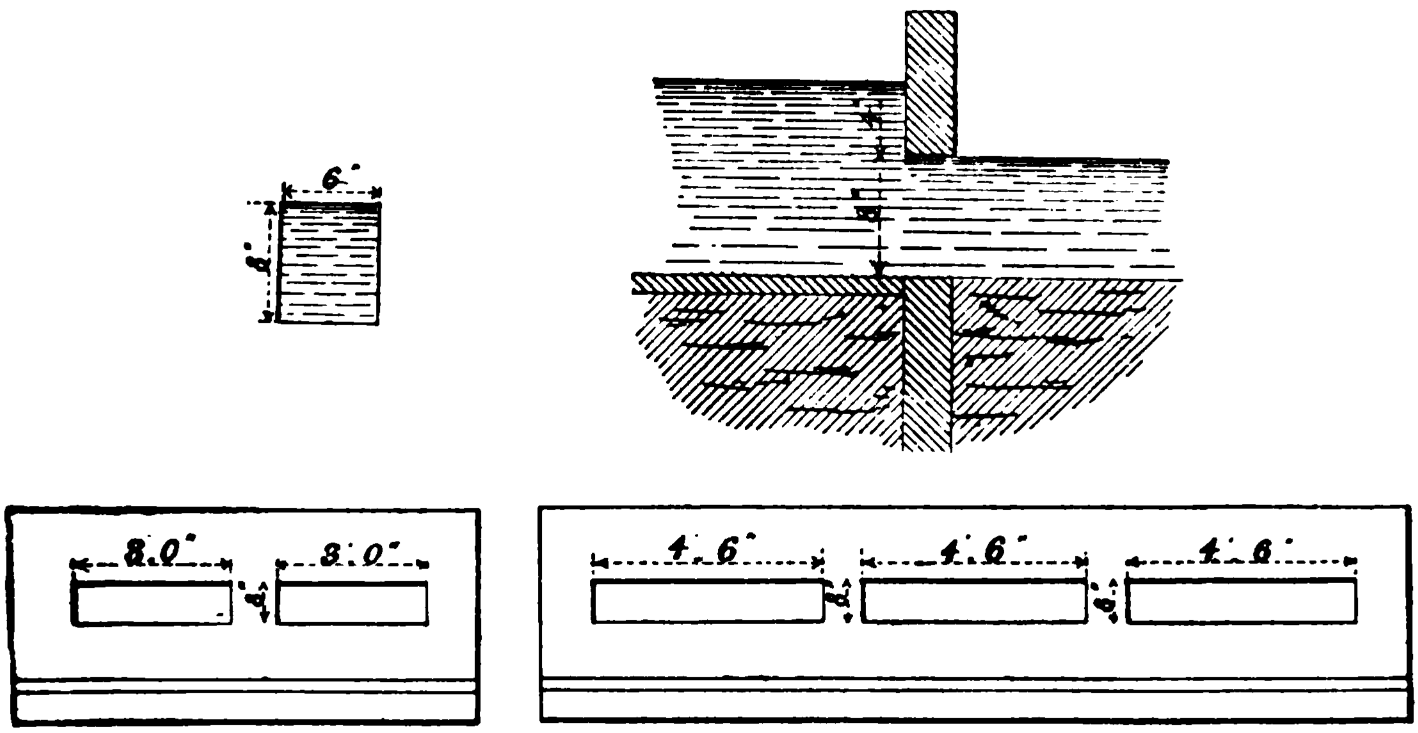
$\frac{1}{330}$, it becomes necessary to retard the velocity of the stream by a series of cascades or dams, for there are few soils that could resist its denuding effects under such circumstances; and if the irrigation take place in a plain where the river has become tolerably clear, the inclination may be made as stated above from $\frac{1}{1800}$ to $\frac{1}{3600}$.

In setting out the main conductor, it is important that the radius of curvature of the changes of direction be made as large as possible, in order to avoid any diminution in the velocity of the flow and the rate of discharge, and also to obviate any destructive action upon the banks. The minimum radius should be from 100 to 150 yards. The banks should be kept at least 8 inches above the water line when the supply is constant; and it is even desirable to make that height from 16 to 18 inches, to guard against any inconvenience from the development of aquatic plants, which takes place with most extraordinary rapidity in all such positions. The peculiar mode of growth of these plants, in long festoons, also produces a greater interference with the rate of discharge than would arise from their precise volume; because they retard the velocity of flow, on account of the manner in which their long streamers follow the direction of the current. It is important that they should be cut as often as possible.

In England the supply of water is usually so copious, that it is rarely necessary to measure the quantity distributed at any particular place. In warmer climates, or even here when the preliminary expense of procuring the water has been considerable, its economical value becomes, however, so much enhanced that it is a matter of primary importance to ascertain the quantities supplied to the various recipients. The construction of gauges has, therefore, for a long time occupied the attention of the hydraulic engineers of Northern Italy; and the researches and experiments made by them for the purpose of establishing a simple, self-acting instrument of

that description, have led to the announcement of the curious law of hydrodynamics, not before observed, to which attention has been already called, and upon which is based the principle of the gauges used in Piedmont and Lombardy. A description of these gauges is subjoined, as they may frequently be required in our colonies, or in India.

The unity adopted in the measurement of water in Italy is called *l'oncia d'acqua*, and it is the quantity which could flow through a rectangular orifice, discharging freely at the lower end, but not entirely into the air, under a constant pressure of four inches above the orifice. When it is desired to distribute more than a single ounce, the width only is modified, whilst all the other conditions are retained. The orifices of



Figs. 312.—Measurement of Water, Italy.

discharge are formed of the hardest description of stones to be found in the country, or occasionally of cast or wrought iron, and are cut square without any bevel, or the addition of anything like a funnel capable of facilitating the discharge. There are no prescriptions as to the thickness, which under these circumstances is regulated by the width of the opening; and this latter dimension is usually made of the width necessary to pass six ounces; when more than six ounces are

required to be passed, the number of orifices is increased. The conductor is formed upon the banks of the canal leading from the main stream, by means of wing walls of masonry, and the sill is usually placed at the floor line. If the ground be of a soft or yielding nature, the portion exposed to the wash of the water must be paved, especially in the part where a species of cataract will exist. The opening of the conductor *a b* of Fig. 313 is made equal in width to that of

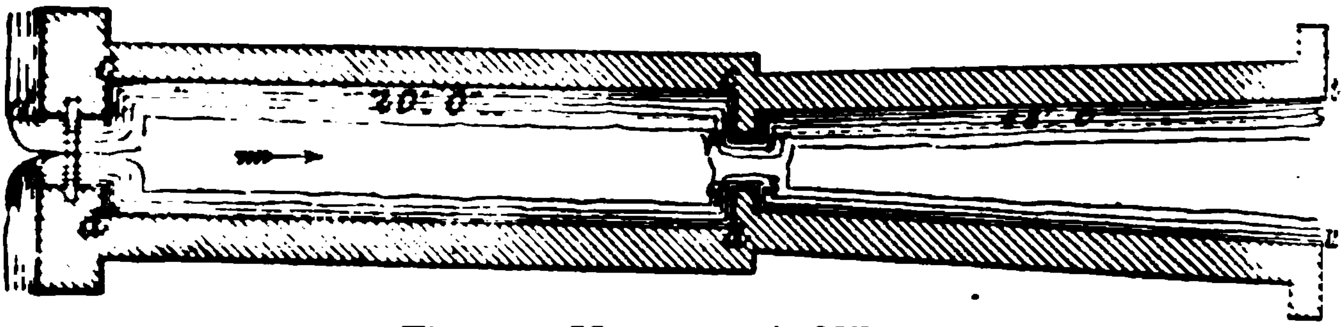


Fig 313.—Measurement of Water.

the orifice of discharge, but the height is not limited. The rectangular space *c c*, *d d*, is made about 20 feet in length, and 10 inches wider on each side than the orifice of discharge, and the floor of this space is laid with a rise of 16 inches in the total length, towards the orifice *g h*. At the level *c d* of Fig. 314 is a flooring, placed for the double purpose of

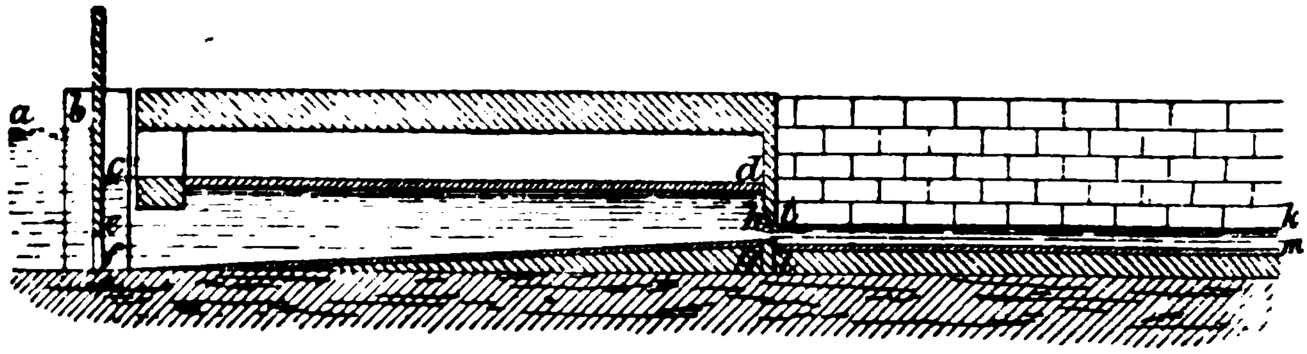


Fig. 314.—Measurement of Water.

preventing the water from rising beyond the prescribed height, and for preventing any movement or agitation on its surface. The entry to this covered portion of the gauge is formed by a stone lintel, the underside of which is exactly level with the top of the orifice, and consequently 4 inches below the surface of the water; and as the height of the orifice is always 8 inches, and the rise of the inclined plane

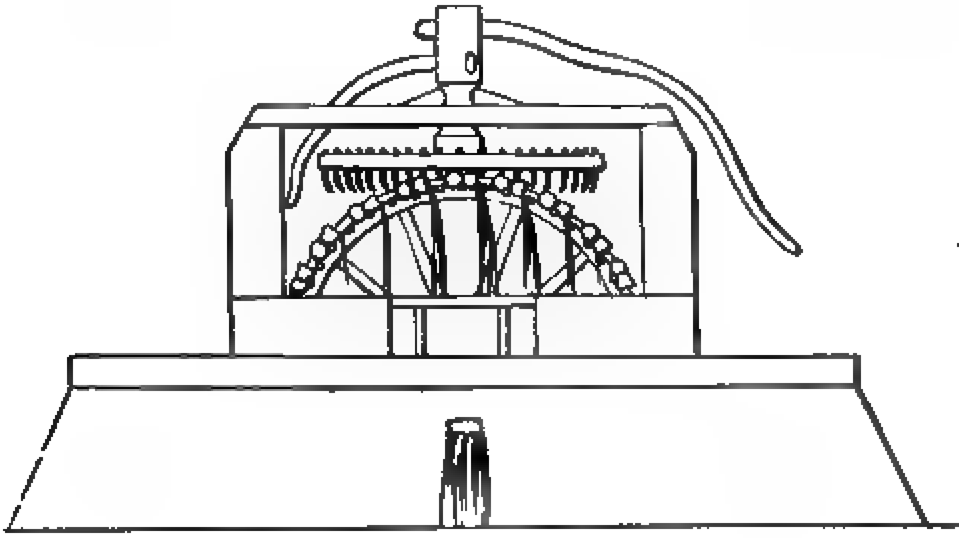
is 16 inches, the underside of this lintel is necessarily 2 feet above the sill of the sluice. Immediately beyond the orifice is the tail chamber, which is made 4 inches on each side wider than the orifice; its length is usually 18 feet, and at the further extremity its width is made 6 inches on each side wider than at the commencement. A small drip of 2 inches is formed at the commencement of the tail bay, and an inclination of 2 inches is given from thence towards the extremity. Gauges of this description require a minimum difference of level of 8 inches between the water on the respective sides of the sluice, and so cannot be applied upon canals with less than 3 feet of water.

It must be evident that a gauge such as is above described is far from being theoretically perfect. Indeed there can be no question but that the interference of the contraction of the fluid vein upon the discharge of a small orifice, must be far greater than that which takes place in a large one; and it has actually been found that the discharge through a single orifice of 6 ounces exceeds that which would take place through six smaller orifices of 1 ounce each, in the ratio of 282 to 222. For all practical purposes, however, the Italian engineers consider these gauges to be sufficiently correct; but they do not allow more than 6 ounces to pass through any one opening.

It is necessary to construct waste weirs and overflows upon the sides of the main conductor, especially when the stream from which the water is supplied is liable to sudden and considerable variations in its volume.

In some parts of France, and in the Milanese territory, a supply of water for irrigation has been obtained from artesian wells; and when the spring which feeds this well rises from a considerable depth, it is, generally speaking, of a very superior quality for the purpose in view. The higher temperature of the waters thus obtained to that of river waters, is of itself an important recommendation in their favour, and

it is indeed one reason why they are principally used in Northern Italy for the "marcite," or winter meadows. At times, also, the mineral elements contained in well waters



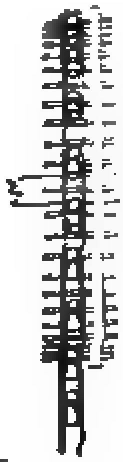


Fig. 315.—Noria.

are of great value; but it is rarely that the volume they furnish is sufficient for an extensive application. In other

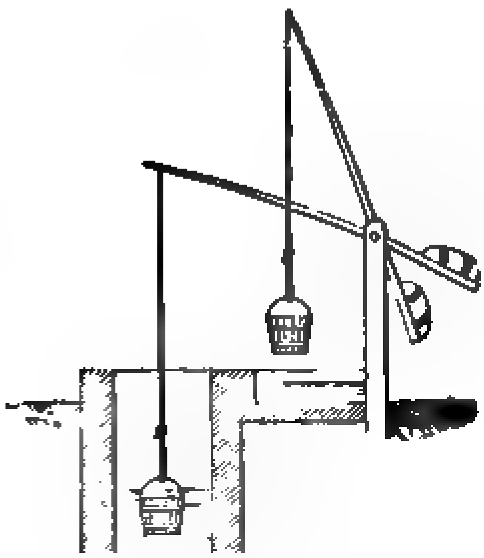
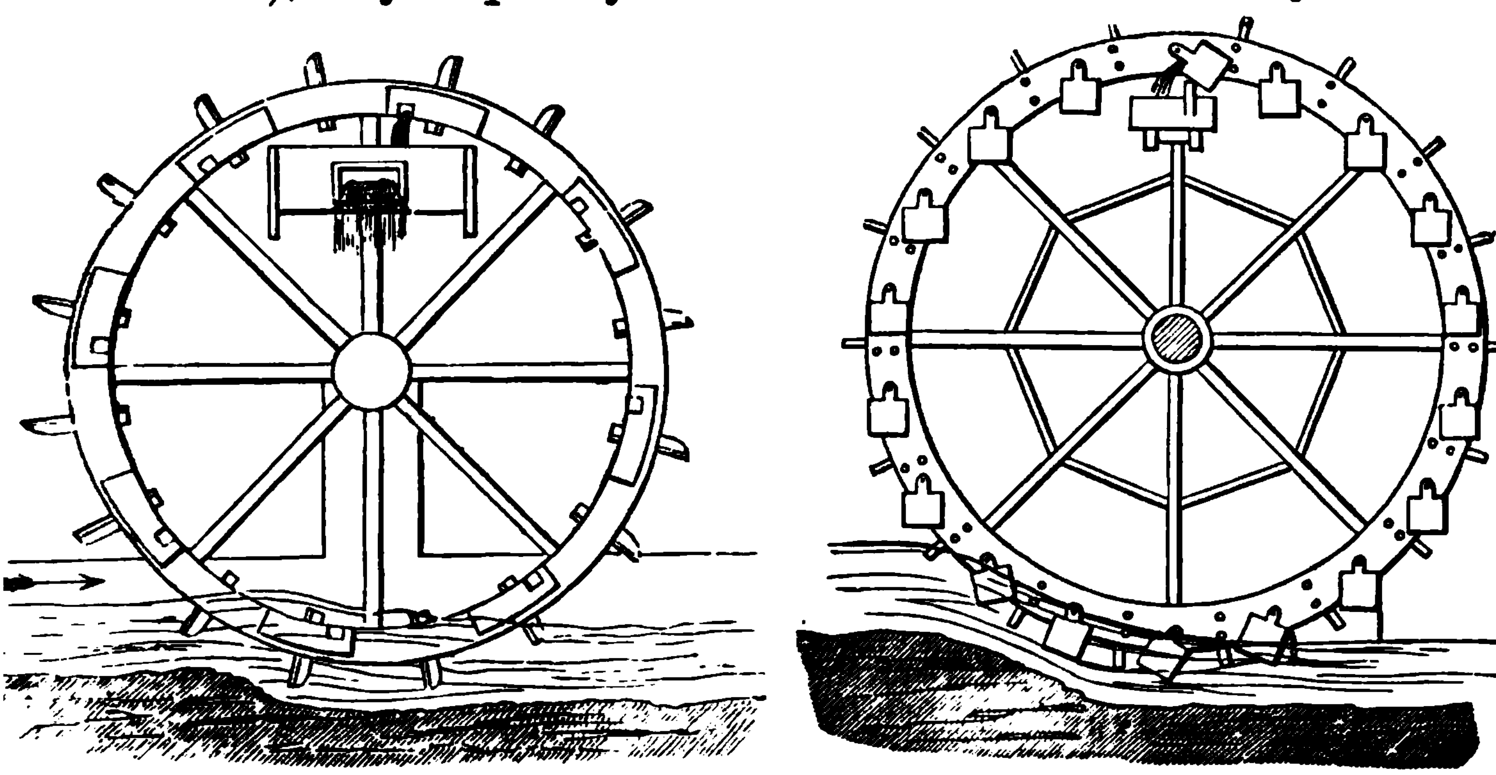


Fig. 316.—Padouf.

Fig. 317.—Persian Wheel.

countries, especially in warm latitudes, mechanical means are resorted to for the purpose of raising the water to the height required; and windmills, norias, (Fig. 315), swapes, or

fadoufs (Fig. 316), Persian or bucket wheels (Figs. 317 and 318), may frequently be seen in motion with that object.



Figs. 318.—Persian Wheels.

The noria is indeed one of the characteristic instruments of the Moorish agriculture, and may be observed in all the countries where the Saracens settled for any length of time; whilst the "fadouf" may be observed in the records of Egyptian civilisation recorded in their temples or hieroglyphical writings. In our own country steam power has been applied for raising drainage waters; but, with the exception of the small works executed at Rugby for the distribution of the town sewerage, the author is not aware of the erection of any steam-engine exclusively for irrigation purposes, though there can be no doubt but that such an application would be highly profitable in many cases.

There is one of those questions of detail which certainly merits more attention than it has hitherto received from our agricultural engineers, namely, whether or no it be necessary to manure the lands to be irrigated? It would appear, from what has been hitherto recorded, that the answer to this question would depend mainly upon the quantity of water to be distributed, upon the relative natures of the soil and of

the waters. The German irrigators, who are able to dispose of large quantities of water, as we also are in England, have a popular proverb to the effect that "he who has water has grass;" but in the north of Italy, where the supply of water is limited, the universal practice is to manure the lands highly before commencing a course of irrigation. In the granitic districts of Northern Spain there does not appear to be any reason for the application of any fertilising ingredients beyond those which are supplied by the water itself; and even in parts of the Campine, or the plains near Antwerp, meadows are known to be annually improved simply by the application of water without the addition of any manure. The grasses in our northern latitudes act, indeed, to convert the mineral and organic matters contained in the waters for their own nourishment; but in warmer latitudes the function discharged by the waters distributed by irrigation is to facilitate the assimilation of the elements required for the growth of the plants, rather than themselves to furnish those elements.

Generally speaking, the turf, or the natural grass surface of a country laid out for irrigation, will suffice for the covering of the ground over which the water is to flow; but as it may occasionally be necessary to sow grasses for the purpose of, as it were, creating a new vegetation, it may be worth while to give a translation of the mixtures of seeds which are recommended by the most practical foreign irrigators for the various descriptions of soils. Thus, for sandy soils, a mixture is recommended composed of the seeds of—

1.	Phelum pratense	2 lbs.
	Agrostis vulgaris	6 „
	Holcus lanatus	4 „
	Poa trivialis	6 „
	Trifolium repens	12 „
	Medicago maculata	3 „
	Lathyrus pratensis	3 „
	Per acre	36 lbs.

2. For a sandy soil with a slight mixture of clay:

Phelum pratense	2 lbs.
Poa trivialis	6 „
Festuca elatior	6 „
Lolium perenne	4 „
Avena pubescens	3 „
Vicia sepium	2 „
Lotus corniculatus	2 „
Trifolium pratensis	10 „
Per acre	35 lbs.

3. For calcareous soils:

Bromus pratensis	5 lbs.
Dactilis glomerata	4 „
Avena elatior	4 „
Lolium perenne	2 „
Poa trivialis	9 „
„ pratensis	2 „
„ augustifolia	2 „
Medicago maculata	2 „
Trifolium pratense	6 „
„ fragiferum	4 „
Per acre	40 lbs.

4. For stiff clayey soils:

Phleum pratense	2 lbs.
Alopecurus pratensis	4 „
Poa trivialis	9 „
Fustaca pratensis	4 „
„ elatior	3 „
Peucedanum officinale	3 „
Medicago maculata	2 „
Trifolium pratense	10 „
Lathyrus pratensis	2 „
Vicia sepium	2 „
Per acre	41 lbs.

Of course it must not be considered that the attempt to fix these proportions is anything more than a rude attempt to fix the composition of the grains to be sown; and every

farmer must exercise his own discretion as to the precise nature of the mixture he will employ. If sowing should be resorted to, it would appear that in our northern parts of Europe the most advantageous period for performing this operation is about the month of March; and, for the purpose of protecting the young plants, it is customary to sow some of the cereal crops at the same time with the grasses. Oats seem to be the most useful in such cases, and they are cut in flower, to be used as fodder; or the buckwheat may be used, provided it be not allowed to shed its grain, for otherwise the new plants would run the risk of being smothered by it.

When water is used, as in the warmer regions of the East, for garden cultivation, the manner of its application must vary essentially from that resorted to in North-Western Europe for meadow lands, on account of the different function it has to perform. In the former case the water principally acts to refresh the vegetation and to facilitate the assimilation of its nourishment, and it is therefore made to infiltrate the ground, instead of flowing over it in a uniform stream, as is the case in water meadows. The intervals between the watering of the irrigated meadows, however, enables that class of operation to be carried on more economically (so far as the mere consumption of water is concerned) than when, as in garden irrigation, the feeding channels must be kept constantly full; and thence it happens that the latter operation is rarely performed when the supply of water is obtained from reservoirs. Wherever in the East a permanent supply has been obtained, the garden cultivation has been applied; and it might almost be said that in those regions irrigated orchards and gardens take the place of our meadows. The dry, clear, burning atmosphere has indeed there rendered irrigation necessary not only for the plants, but also for the comfort of man, and even the worst regular governments have striven to secure that blessing. In the plains of Syria, and in the dominions of the Mohammedan kings of

India, great works have thus been undertaken for this purpose; and, indeed, the engineers of our East India Company have lately had little else to do with the irrigation canals of their predecessors than to repair and slightly redress them, in order to restore their efficiency. There is one difference, however, between the irrigation provided in Syria and that of our Indian possessions, viz., that the former is almost exclusively devoted to garden cultivation, whilst the latter is occasionally applied to the growth of rice, and the enormous quantities of water which the East Indian engineers are able to dispose of have enabled them to combine other commercial applications of water with the one they principally had in view. But whilst dwelling upon this part of the subject, it may be as well to observe, that at all times, and in all climates, the tendency of irrigation is to develop in the plants receiving it a growth of the leaves at the expense of the fruit or grain. This is especially the case in warm climates, where all the operations of nature take place on an extended scale; but the effect of the law is to exclude cereal crops from the system of agriculture in irrigated districts, unless the cereals themselves should be of a peculiar nature, such as the rice, and perhaps also the Indian corn. Moreover, although in India the great feeders for irrigation, canals, are at times made to facilitate a species of canal navigation, and to drive mills, the economical results of such mixed systems have hitherto been more than questionable.

In the warmer latitudes, as has been before observed, water is largely used for the purpose of creating artificial rice-grounds, and the conditions of the growth of that plant, as well as those of the application of water to it, are sufficiently distinct from the conditions which prevail in ordinary irrigation to justify a passing reference to them. Now, the rice is essentially an aquatic plant, and it only grows in latitudes situated below the parallel of 46° north. During its growth it requires to be constantly immersed in water;

and it would seem that the quality of the land upon which it is grown is a matter of far less importance than that of the water employed, and that the water is by so much the more fitted for the irrigation of rice-fields as it is charged with the greater quantity of extraneous matter. For this reason river and pond waters are preferable to spring waters; and, indeed, the coldness and purity of the latter are at times so objectionable in rice-fields, that it is considered necessary to expose them in shallow reservoirs, and to mix them with animal manure before pouring them upon the land. It is usually calculated that the quantity of water required to irrigate a rice-field is about 1 cubic foot per minute, and per acre. This style of cultivation may either be permanent, or it may form part of a rotation: in the first case it is adopted because the land is marshy, either from the want of outfall or from the springs rising in it; in the second, a species of artificial irrigation is required for every crop of rice which is to be raised from the land.

Whatever may be the nature of the ground to be converted into rice-lands, the first condition required is that the water should be kept continually in motion, and that all of it which is brought upon the land should be removed. A series of plane surfaces must thus be formed, so that no part of the land may be left dry, and that the water may not be allowed to stagnate in any part. After the land has been properly levelled, it is to be ploughed, and then the retaining banks are to be formed; of these there are two sorts: 1st, the longitudinal ones, or those which have the same direction as that of the stream, and which are intended to last as long as the field is laid down in rice; and 2nd, the transverse banks, which intercept the current in an angular direction, so that when the banks are completed the rice-field will be divided into a series of polygons. The sizes of these polygons is principally regulated by the difference of levels of the planes themselves; and they are made the

mallest in those cases wherein the inclination is the greatest, n order to economise the labour of disposing them in norizontal planes. Moreover, the dimensions of these ields are limited by the consideration that the larger they re, the greater probability there must be that the wind may ear up the young plants. It is usual to make the banks

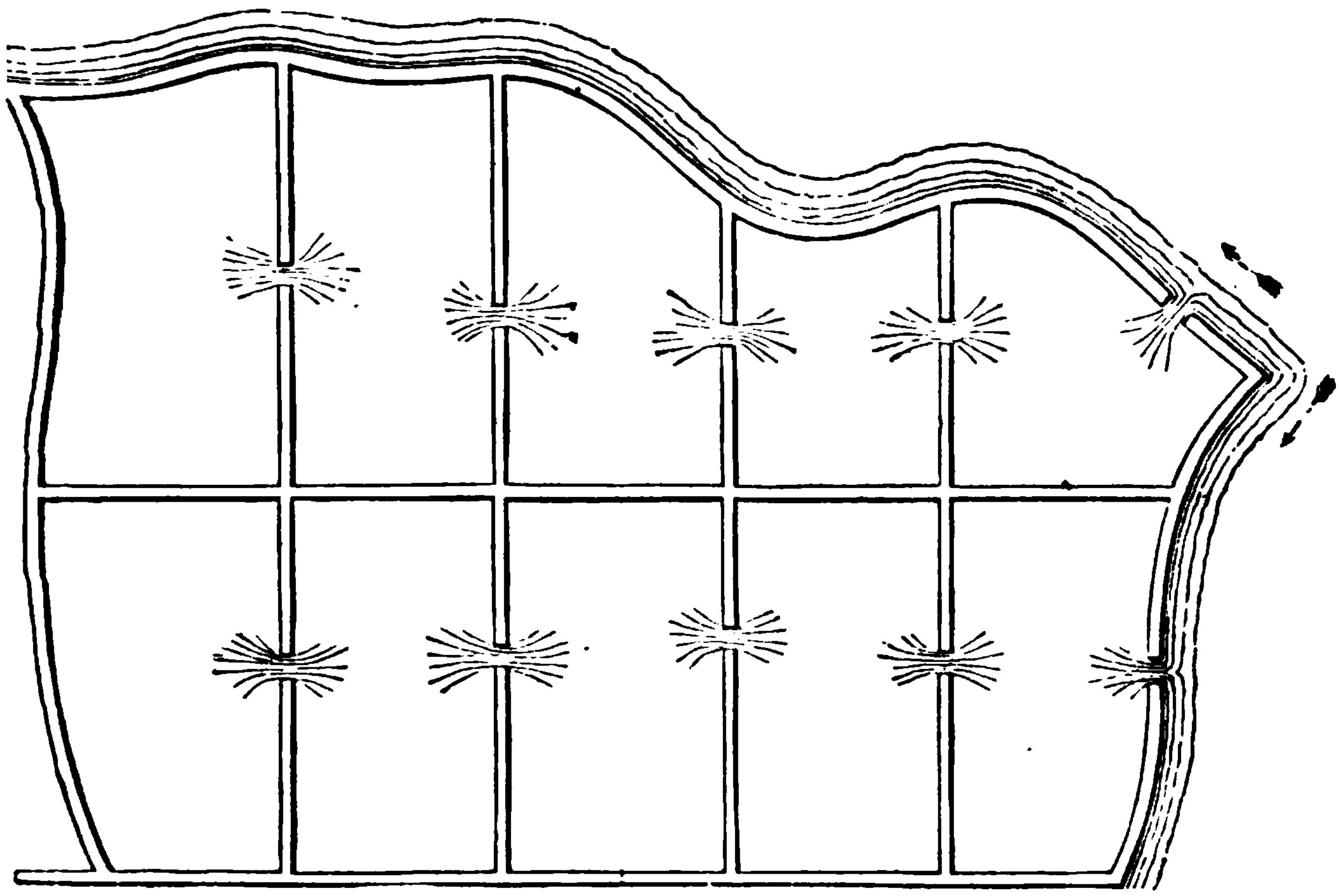


Fig. 319.—Irrigation of Rice Lands.

about 6 inches above the ground on the upper side of the field, and about 2 feet above that level on the lower side; the width is never less than 6 inches at the crown; but as the top of the bank often serves for a road, as well as for the immediate object of their formation, the width may vary indefinitely. They are made with the earth taken from the lower parts of the field; and when they are roughly terminated, the water is let into the first division and allowed to rise about 5 inches all over the surface. Openings are then made in the lower banks, and water is successively let into them, so that, in fact, the whole of the field is converted into a succession of small ponds, separated by the several

banks. During the whole of the growth of th
thus exposed to be irrigated by flooding; but th
the manner of this flooding will vary with the h
plant, its degree of maturity, and the violence
It becomes, therefore, necessary to regulate the a
the water in such a manner as to be able to cont
at any moment, and even occasionally to shut it o
After the rice crop has been carried, all the wat
drawn, and the land is left exposed to the act
atmosphere throughout the winter, and until the sp

In the Humber district it is found that the warp
are at first cold and raw, and that they require a
treatment for agricultural purposes. Thus they
favourable for the growth of corn; oats may succe
them, but barley never will. The rotation usually
is as follow:—The new warp is sown with grass
years; on the third year wheat is sown; on the
beans; on the fifth, wheat again. Should the groun
warped be found to contain too much salt, it must be ex
to the air for some time before being brought into c
tion; and at all periods it is found to be objectional
allow the salt warp to deposit upon growing grasses. In
in Yorkshire, it is customary to let the newly-warped
lie fallow for twelve months before sowing the grass,
to let on the waters after the second crop of wheat
been raised.

There is a system which acts principally by infiltration, a
is applied in hilly districts as much for the purpose of o
viating any ravinement, so to speak, of the vegetable soils (
their inclined slopes, as it is for the purpose of irrigatio
strictly speaking. The feeders are in this case made a
horizontal as possible, and the banks are raised, so that th
water shall not flow over the sides; but it is allowed to per-
meate the soil in a manner dependent of course upon the cha-
racter of the latter. In Devonshire, &c., as was before said,

modification of this system is adopted, under the name of he *catch-water meadows*, which consists in allowing the water o flow over the edge of the lower sides of the feeders in a mall shallow stream, to be collected in a series of parallel ower horizontal feeders which retard its velocity, and retain any vegetable or alluvial matters the waters might remove. A drain is usually carried from the top to the bottom of a meadow of this description, at right angles to the feeders, for the purpose of removing the water from them if required; but the entrances to these drains are closed when the irrigation is to be effected. Catch-water irrigation, it may be added, is executed at a much cheaper rate than any other. For it is usually calculated that the first cost of laying down any large area on a system of bed-work irrigation is about £10 per acre, whilst that of a system of catch-water irrigation is only about £5 per acre. In the case of the Duke of Portland's celebrated water meadows at Mansfield, the total outlay was not less than £30 per acre; but as it is tolerably well known that the enhanced value of irrigated land, as compared with ordinary land, is not less than from £1 10s. to £2 per acre, it is strange that so little attention should at the present day be paid to the subject. In India, the irrigation works have yielded at least from 40 to 60 per cent. on the outlay; and though we cannot expect in England to obtain equally brilliant results, there is no reason to doubt but that operations of this description would still be eminently successful. The irrigation of the barren sands of the Campine by the waste waters of the canal from the Meuse to the Scheldt, it would have been supposed, would have induced the persons interested in the suffering canal property of England to examine whether the sale of their waste waters might not compensate to them in some manner for the destruction of their carrying trade by the railways. The old Dutch engineers, who designed the irrigation of the valley of the Itchen, in Hampshire, made a very creditable attempt

to apply a mixed system of canal and irrigation works: creditable, that is to say, when the state of the science of applied hydraulics in their day is taken into account; and, not to leave the county of Hampshire itself, it must appear strange that the Basingstoke Canal proprietors have not attempted to apply the lesson they might have learnt from their predecessors.

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